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PHASE I INSPECTION REPORT. NATIONAL DAM SAFETY PROGRAM. EAST LA--ETC(U)
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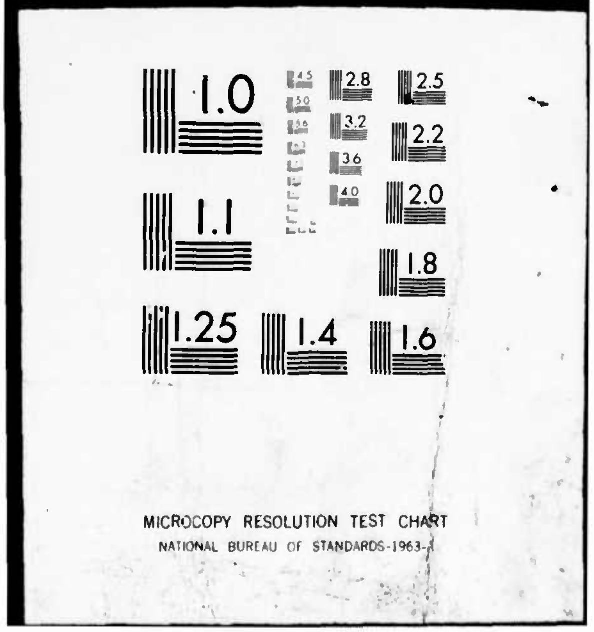
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LEVEL II

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DELAWARE RIVER BASIN

INDIAN FIELDS BRANCH, CUMBERLAND COUNTY

NEW JERSEY

EAST LAKE DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NJ 00070

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
MARCH 1978

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National Dam Safety Program
Dam Inspection Report Phase I
East Lake Dam, N.J.
Dams - N.J.

ABSTRACT

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

410 760

ABSTRACT

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

16 JUN 1978

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, NJ 08621

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for East Lake Dam in Cumberland County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on pages 1 and 2 of the report.

The inspection indicates the dam's earth embankment to be in poor condition and the dam's spillways and outlet channel to be seriously inadequate. To insure adequacy of the facility, the following actions, as a minimum, are recommended:

a. Hydraulic and hydrological investigations should be initiated, within three months after the date of approval of this report, to determine required corrective action(s) necessary to substantially improve hydraulic capacities of the combined spillway facilities and outlet channel. Construction of improved spillway and outlet facilities should be completed in calendar year 1979.

b. Within four months from the date of approval of this report, engineering studies should be initiated to determine a positive means of controlling seepage through the dam, adequate upstream face slope protection and the method of installing an outlet pipe at the low point of the reservoir. Implementation of the results of these studies should be initiated in calendar year 1979.

c. Within one year from the date of approval of this report, the road surface drainage problem on top of the dam should be rectified, the wall on the upstream side of the roadway repaired and the settled areas atop the dam properly filled.

NAPEN-D
Honorable Brendan T. Byrne

Two copies of the report are being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Edwin B. Forsythe of the Second District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, thirty days after the date of this letter.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingl[y] request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

1 Incl
As stated

Cy Furn: w/incl (dupe)
Mr. Dirk C. Hofman, P.E.
Department of Envirnmental Protection

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COHANSEY RIVER BASIN

Name of Dam: East Lake Dam
County and State: Cumberland County, State of New Jersey
Inventory Number: 00070

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

Prepared by: O'Brien & Gere Engineers, Inc.
Justin & Courtney Division

For: United States Army Corps of Engineers
Philadelphia District

Date: March 20, 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam East Lake Dam

State Located New Jersey
County Located Cumberland County
Stream Mill Creek
Date of Inspection February 3, 1978

ASSESSMENT OF GENERAL CONDITIONS

The East Lake Dam is an earth fill structure consisting of poorly compacted random fill and organic material. Settlement is visible in the sidewalk on top of the dam. The upstream slope of the dam is not properly protected against erosion and wave action. Seepage at the downstream slope has been observed in the past by the owners of the fabric plant located immediately downstream of the dam. To correct this problem, a foundation subdrain system and sump pit were installed. The surface drainage system for the road on top of the dam is inadequate. The combined spillway facilities are unable to pass one-half of the Probable Maximum Flood (PMF) without overtopping the dam. The dam is not equipped with a means of draining the bottom 8 feet of the reservoir below the principal spillway crest.

Because of the deficiencies listed above, a further investigation, as outlined in the National Program of Inspection of Dams, Volume I, Appendix D, Chapter 4, is recommended. The above deficiencies are believed to be serious and the further investigation should begin as soon as possible.

The dam has also been investigated previously by another private consultant. One of the results of that investigation was that the State of New Jersey Department of Environmental Protection directed the dam operators to lower the normal reservoir elevation by 4 feet by keeping the principal spillway gates open. Other improvements are apparently being considered for this dam but the details of the investigation or the status of the proposed improvements were unavailable due to pending litigation.

The inspection indicates the dam's earth embankment to be in poor condition and the dam's spillways and outlet channel to be seriously inadequate. To insure adequacy of the facility, the following actions, as a minimum, are recommended:

a. Hydraulic and hydrological investigations should be initiated, within three months after the date of approval of this report, to determine required corrective action(s) necessary to substantially improve hydraulic capacities of the combined spillway facilities and outlet channel. Construction of impr spillway and outlet facilities should be completed in calendar year 1979.

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c. Within one year from the date of approval of this report, the road su face drainage problem on top of the dam should be rectified, the wall on the upstream side of the roadway repaired and the settled areas atop the dam properly filled.

APPROVED: Harry V Dutchyshyn
HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

DATE: 16 June 1978

UPSTREAM FACE OF EMBANKMENT LOOKING WEST

TOP OF DAM LOOKING WEST—UPSTREAM SIDE

TABLE OF CONTENTS

TEXT		PAGE
Assessment of General Conditions		
Overall View of Dam		
Section 1	Project Information	1 - 4
Section 2	Engineering Data	5
Section 3	Visual Inspection	6 - 7
Section 4	Operational Procedures	8
Section 5	Hydraulic/Hydrologic	9
Section 6	Structural Stability	10
Section 7	Assessment/Remedial Measures	11 - 12

FIGURES

Figure 1	Vicinity Map
Figure 2	Drainage Basin Map
Figures 3 - 6	Drawings from "Study to Improve Capacity and Safety of East Lake & East Commerce Street Dam"
Figure 6A	Sketch of Project Features
Figures 7 - 14	Soil Boring Logs
Figure 15	Geologic Map

APPENDIX

Field Inspection Report	A1 - A8
Photographs	A9 - A12
Hydraulic & Hydrologic Calculations	A13 - A32
HEC-1 Computations	A33 - A43

Recommended Guidelines for Safety Inspection of Dams, Chapter 4

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM EAST LAKE DAM ID# 00070

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. This report is authorized by the Dam Inspection Act, Public Law 92-367, and has been prepared in accordance with contract #DACW61-78-C-0052 between O'Brien & Gere Engineers, Inc., Justin & Courtney Division, and the United States Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection. The purpose of this inspection is to evaluate the structural and hydraulic condition of the East Lake Dam and appurtenant structures, and to determine if the dam constitutes a hazard to human life or property.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances. East Lake Dam is an earth embankment with a maximum height of about 17 feet and a length of about 500 feet. East Commerce Street, an asphalt paved, two lane road, is located on the top of the embankment. The top of the embankment varies from elevation 26.5 feet Mean Sea Level (MSL) at the east end, to elevation 24.0 feet MSL at the west end. A masonry wall has been constructed along the upstream side of the road to elevation 27.5. The elevation of the base of this masonry wall could not be determined by visual inspection or from available data. The Murbeck Knitted Fabrics plant is located immediately downstream of the embankment. This plant uses water from the reservoir for fabric processing.

The principal spillway is located on the east side of the embankment, and consists of two gated 48 inch circular openings on the upstream face of a concrete box structure. Rectangular overflow openings are located on the two sides of the box structure about 4 feet above the invert of the circular openings. Outflow from the chamber is through two 48 inch cast iron pipes through the embankment. The downstream invert of the pipes is 5 feet above a concrete basin used to direct flow into a box culvert.

An 11 foot wide emergency spillway is located along the west bank of the reservoir about 150 feet upstream of the embankment. Several weirs and culverts are located along the spillway channel downstream.

An intake structure and pump are located in the reservoir near the west end of the embankment to supply water to a constant head tank for use by the Murbeck plant. Excess water flows through a return line back into the reservoir.

b. Location. The East Lake Dam is located in the city of Bridgeton, Cumberland County, New Jersey. It is located along East Commerce Street, just downstream of the confluence of Indian Fields Branch and Jackson Run. Flow from the spillway channel discharges into Mill Creek about 3,000 feet above the Cohansey River. Mill Creek is subject to tidal variations. See the vicinity and Drainage Basin Maps in the appendix (Figures 1 and 2).

c. Size Classification. The maximum height of the dam (to the top of the wall) is about 17 feet and the maximum storage capacity of the reservoir is about 230 acre-feet. Therefore, the dam is in the small size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification. A large fabric plant is located immediately downstream of the dam, with business and residential areas of Bridgeton within a mile downstream of the structure. Therefore, the dam is in the high hazard category as defined by the Recommended Guidelines for Safety Inspection of Dams.

e. Ownership. The ownership of the dam is in question. The dam may have been originally a privately owned structure. The City of Bridgeton assumed the maintenance responsibility for the road on top of the dam. However, neither the City of Bridgeton nor Murbeck Fabrics have accepted the responsibility for the dam itself.

f. Purpose of Dam. The dam is used to store water for use in fabric processing at the Murbeck Fabric plant downstream of the dam.

g. Design and Construction History. No information is available on the dam design or the construction methods used. The dam is reported to have been built in 1805.

Recent borings show that the dam was constructed of various types of earth and organic materials. It is doubtful that proper placement and compaction techniques were used to construct the dam.

h. Normal Operational Procedures. See Section 4 for information on operation procedures.

1.3 PERTINENT DATA

a. Drainage Areas. The East Lake Dam is located just below the confluence of Jackson Run and Indian Fields Branch. These streams drain areas of 1.4 square miles and 5.9 square miles respectively. The total drainage area above the dam is 7.3 square miles.

b. Discharge at Damsite. No discharge records are available at this site. According to the plant engineer at the Murbeck Fabric plant, both the principal and the emergency spillways were inadequate to prevent property damage adjacent to the dam during tropical storm Doria of 1972. East Commerce Street was flooded near its junction with East Avenue during this flood. The worst condition occured at high tide, when the box culvert below the principal spillway flowed under pressure, causing backup of flow through the culvert manholes in the fabric plant buildings downstream of the dam. Water in the plant reached a depth of about 10 inches above the floor during this flood.

The capacity of the principal spillway is 230 cfs when the reservoir is at the top of the wall on top of the dam. The capacity of the emergency spillway is limited by weirs, bridges, and culverts under East Commerce Street. The maximum flow in the emergency spillway, before general flooding of the surrounding area occurs, is estimated to be 20 cfs.

c. Elevations above MSL. (from Figure 4)
Top of stone wall - 27.5 feet
Top of dam - varies between elevation 24.0 and 26.5
Principal spillway crest (invert elevation of gated openings) - 20.5 feet
Design Surcharge - 29.9 feet ($\frac{1}{2}$ PMF)
NOTE: The design flood overtops the wall by 2.4 feet

d. Reservoir
Length of normal pool - 1800 feet (elevation 20.5)
Length of maximum pool - 2600 feet (elevation 27.5)

e. Storage (from Figure 6)
Normal pool - 75 acre-feet (elevation 20.5)
Maximum pool - 230 acre-feet (elevation 27.5)

f. Reservoir Surface Area (from Figure 6)
Normal pool - 17 acres (elevation 20.5)
Maximum pool - 26.5 acres (elevation 27.5)

g. Dam

Type - earth fill
Length - 500 feet
Top width - 60 feet
Side slopes - 2 horizontal : 1 vertical (estimated)
Zoning - unknown
Impervious core - unknown
Cutoff - unknown
Grout curtain - unknown

h. Diversion and Regulating Tunnel

None

i. Spillways (Elevation taken from drawings supplied)

1) Principal spillway
Two 48 inch cast iron pipes
Length of weir - 8 feet
Crest elevation - pipes, 20.5 feet MSL - weir, 24.5 feet MSL
Gates - two 48 inch circular gates
U/S Channel - none
D/S Channel - concrete box culvert; 5.8 foot wide, 5.66 foot deep

2) Emergency spillway
Type - earth channel with stone and masonry walls
Length of weir - 11 feet
Crest elevation - 24.3 feet MSL
Gates - none
D/S Channel - concrete culvert under East Commerce Street discharges into a sluiceway under the Murbeck Fabric plant.

j. Regulating Outlets. A pump is used to supply water through a pipe in the embankment to a constant head tank downstream of the dam for use in the fabric plant. Excess water from the tank is returned to the reservoir through an overflow pipe.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN - The information available for review of the East Lake Dam included:

a. Soil boring logs from tests taken in November of 1974. A copy of the logs is included in the appendix (see Figures 7 through 14).

b. Seven drawings from a "Study to Improve the Capacity and Safety of East Lake and East Commerce Street Dam," by A.G. Lichtenstein and Associates, Consulting Engineers, Teaneck, New Jersey. The study was jointly financed by Murbeck Knitted Fabrics, Cumberland County, and the City of Bridgeton. Pertinent drawings are included in the appendix. The study was not available for inspection, due to pending litigation concerning ownership of the dam.

2.2 CONSTRUCTION - No information on construction materials or techniques is available.

2.3 OPERATION - See Section 4.

2.4 EVALUATION - The soil borings show that the dam consists of sand, gravel and silt. The number of blows on the 1-3/8 inch diameter core sampler, with a 140 pound hammer dropping 30 inches, varied from 1 to about 20 per 6 inches of penetration. Organic material, ashes, cinders, wood chips, sawdust and peat were all encountered in the dam or foundation. The groundwater was at about elevation 11.0 in most of the borings after 25 days, although it was as high as elevation 14.2 in boring B9. The ground downstream of the dam varies from elevation 13 to elevation 17.7 and groundwater was between 2.1 and 3.5 feet from the ground surface in the area. Without the subdrain system in the foundation downstream of the dam, this level would undoubtedly be higher. The borings indicate that the dam is composed of loose fill of medium to high permeability as well as compressible material. The drawings by A.G. Lichtenstein show existing topography and some proposed improvements. They show little detail with regard to the composition or condition of the existing facilities. Evaluation of these plans is beyond the scope of this investigation.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. Dam. The dam is an earth embankment with a stone wall on the upstream side of the crest. The stone wall is noticably settled and tilted. The upstream slope consists of a loose sand fill which shows evidence of erosion and settlement. Vertical depressions of 2 to 3 feet are evident near the water surface. There are no training walls at the outlet structure to protect against the effects of erosion in this area. Erosion to depths of 3 to 4 feet is apparent for a length of 30 feet west and 50 feet east of the outlet structure. Sand has been placed in these areas to protect the embankment.

The profile of East Commerce Street on the top of the embankment is such that ponding occurs near the west end of the embankment during periods of rainfall excess. The roadway has no effective surface drainage system in the area of the dam, so the ponded water percolates into the embankment. There are two areas of excessive settlement along the sidewalk on the downstream side of East Commerce Street. These areas are about 30 and 40 feet long, 3 feet wide, with settlement from 1.5 to 2.5 feet. A similar settlement area about 30 feet by 3 feet by 9 inches deep is located near the middle of the dam on the upstream side of the road. It was reported by the plant engineer that the water entering the settlement areas appears along the downstream face and toe as seepage. In 1955, the plant installed French drains at the toe of the dam to remove the seepage water and to lower the water table. A pump used to remove water from the drains, keeps the water table about 6 inches below the floor of the plant. The extent of the drainage system is not known. No seepage was observed coming from the downstream slope of this embankment during the inspection.

b. Appurtenant Structures. The principal spillway consists of two 48 inch gated openings leading to a concrete box structure. Overflow openings are located on the sides of the box structure, 4 feet above the gated openings. Outflow from the box structure is through two 48 inch cast iron pipes lined at the entrances with 42 inch corrugated metal pipes. (Refer to the sketch on page A28.) The gates, which are kept open, are operated from the top of the box structure. The concrete in this structure is deteriorated. The gates are badly rusted but, according to Murbeck's Plant Engineer, are operational. Concrete has spalled in a number of places, the columns are cracked, and there is a large crack near the junction of the beam with the slab. Repair work has been done to the downstream side of the box structure.

The emergency spillway is an earth channel about 11 feet wide constructed through the grounds of a private estate. Flow travels 400 feet before entering 18 inch and 24 inch diameter pipes under East Commerce Street. Channel obstructions include 3 bridges and 2 weirs. The ability of the emergency spillway to discharge flood waters is limited by the small culverts and obstructions along the channel.

Water is pumped to a constant head tank located in the downstream slope of the embankment through a pipe in the embankment. The condition of the pipe through the embankment was not determined during the inspection due to inaccessability.

c. Reservoir Area. The emergency spillway channel is constructed through a private estate. This area was flooded in 1972. The conditions in the reservoir area do not appear to affect the safety of the dam.

d. Downstream Channel. The downstream channels are constricted, and constructed under the plant buildings. Mill Creek is a tidal stream, which can have a large affect upon the discharge capacity of the channels.

3.2 EVALUATION Visual inspection of the site shows that the dam and appurtenant structures were not constructed according to modern standards, nor are they being maintained properly. The upstream face of the dam is not suitably protected and maintained. Areas of settlement on the dam may be due to the lack of proper surface drainage or internal erosion (piping). The pipes through the embankment were not inspected throughout their length. However, the readily visible portions appeared to be in good condition. The masonry wall on the downstream side of the outlet works appeared to be in good condition. A further discussion and evaluation of these subjects is included in Section 7.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES - According to the information obtained during the field inspection from the Murbeck Plant Manager and the Plant Engineer, the operational procedures in connection with the dam consist primarily of pumping water from the reservoir into the constant head tank downstream of the dam and the operation of the foundation subdrain system downstream of the dam. Maintenance of the dam has been minimal and appears to have consisted only of sandbagging or dumping sand on the upstream face of the dam to repair eroded areas.

A foundation subdrain system and sump pit have been installed downstream of the dam. Water from the sump pit is pumped into a drainage ditch downstream of the dam.

The dam is not equipped with any facilities for drawing down the reservoir below elevation 20.5 A survey of the reservoir bottom made in 1976 shows that the bottom of the reservoir is at elevation 10.0. The elevation of the ground downstream of the dam is 13.0.

In 1973, after an investigation of the dam was made by a private consultant, the State of New Jersey Department of Environmental Protection (NJDEP) directed the lowering of the normal reservoir elevation by 4 feet. This was accomplished by opening the 48 inch gates in the principal spillway. These gates must be kept open to maintain the reduced reservoir elevation required by NJDEP.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. The Probable Maximum Flood (PMF) hydrograph was calculated from the Probable Maximum Precipitation using standard reduction factors. The Soil Conservation Service curvilinear unit hydrograph was developed for the drainage basin and used as a basis for constructing the PMF hydrograph. Peak inflow discharges to the reservoir for the PMF and one half of the PMF are 25,000 cfs and 12,500 cfs respectively. These inflow hydrographs were routed through the reservoir by the modified-Puls method, utilizing computer program HEC-1. The peak outflow discharges for the PMF and one half of the PMF are 25,000 cfs and 12,450 cfs respectively. Both the PMF and one half of the PMF result in overtopping of the dam.

The stage-outflow relation for the principal spillway was based upon the available information on invert elevations and sizes for the gated openings and the outlet conduits. Further assumptions on discharge through this outlet are noted on page A28. The emergency spillway discharge is limited by the small pipes used to convey flow under Commerce Street. The capacity of the pipes is estimated at 20 cfs under 2 feet of head. At this elevation, a 70 foot section of the wall along East Commerce Street is inundated.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY - East Lake Dam is constructed on unconsolidated sediment of the Bridgeton formation (quaternary) and possibly the Cohansey formation (tertiary). These geological units are composed of fine to coarse grained sand and gravel and contain some thin zones of finer grained sediments (silt and clay). The differences in the two formations are subtle and no distinction can be accurately made at the dam site or from boring data.

The dam is in seismic risk zone 1 of the Seismic Zone Map of the United States. Due to the low height of the dam, the risk of seismic damage is probably low. However, due to the character of the embankment materials as evidenced by the borings, the possibility of liquefaction during an earthquake may exist.

The physical condition of the dam is poor. Considerable settlement, erosion and possibly piping of the embankment has taken place. The stability of the upstream slope is suspect due to the existence of a loose sand fill and the tilted wall. The risk of internal erosion of the embankment and the foundation is high. The dam and abutments are also highly susceptible to surface erosion although the paved roadway will offer some resistance.

Shrub and tree growth on the downstream slope of the embankment makes it difficult to tell if sloughing of the slope is taking place. No leakage was observed flowing out of the embankment slopes; however, this may be because the drainage is controlled by the foundation subdrain system downstream of the dam. The fact that the top of the dam is over 60 feet wide also has a great influence in controlling the seepage and reduces the danger of slope or piping problems. However, these possibilities still exist.

The structural condition of the principal spillway is also questionable. The concrete in the outlet structure showed evidence of substantial cracking and deterioration.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 <u>DAM ASSESSMENT</u> - The East Lake Dam is deficient in the following respects:

a. The embankment contains poorly compacted, pervious soil and organic material. These materials are unable to control the embankment seepage and settlement properly.

b. The embankment has inadequate slope protection on the upstream face.

c. The surface drainage system for the road on top of the dam is inadequate.

d. The dam is not equipped with facilities to empty the reservoir in an emergency.

All of the above deficiencies are potentially dangerous. However, the dam has withstood the test of time without serious consequences. This may be due to the low head to which the embankment is subjected, the unusually wide top width of the dam, the subdrain system downstream of the dam and other factors which cannot be evaluated within the scope of a Phase I Investigation.

In addition to the deficiencies in the embankment listed above, the spillways are inadequate. For a flood of one half of the PMF, the wall on top of the dam is overtopped by 2.4 feet.

For the above reasons, an additional investigation as outlined in the <u>National Program of Inspection of Dams</u>, Volume I, Appendix D, Chapter 4, is recommended to be made of the East Lake Dam as soon as possible.

7.2 <u>REMEDIAL MEASURES</u> - The following remedial measures could be considered for improving the safety of the dam:

a. Construction of a positive means for controlling seepage through the dam such as:

1) An impervious earth core with properly designed filters constructed by conventional or slurry trench methods.

2) An impervious liner on the upstream face.

3) Steel sheet piling.

b. Addition of slope protection for the upstream face (riprap with a filter blanket or equivalent).

c. Filling of settled areas with compacted earth material and improvement of the surface drainage system.

d. Repair of wall on the upstream side of the roadway.

e. Construction of a new spillway.

f. Installation of a pipe at the low point of the reservoir for drainage purposes.

FIGURES

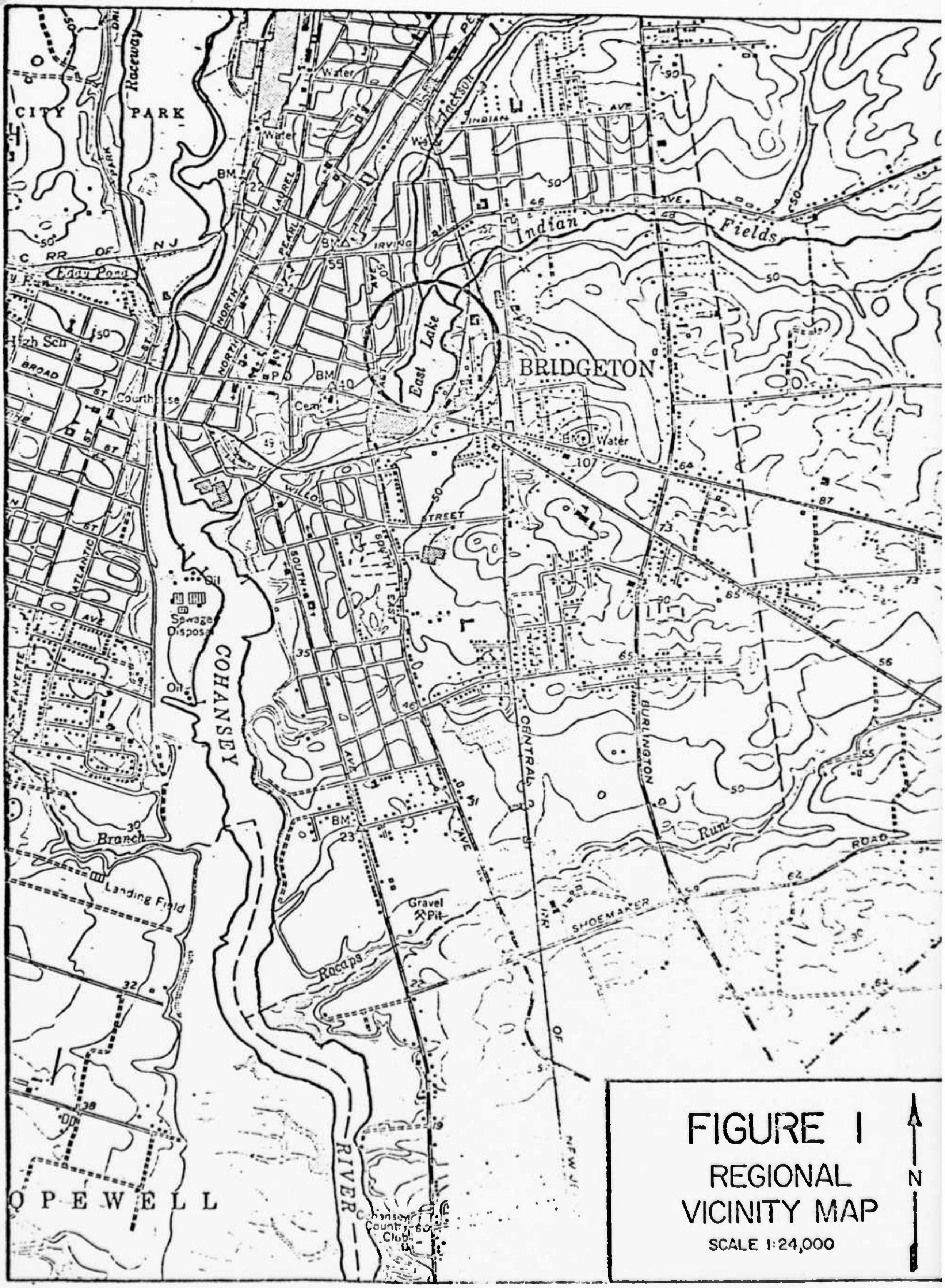
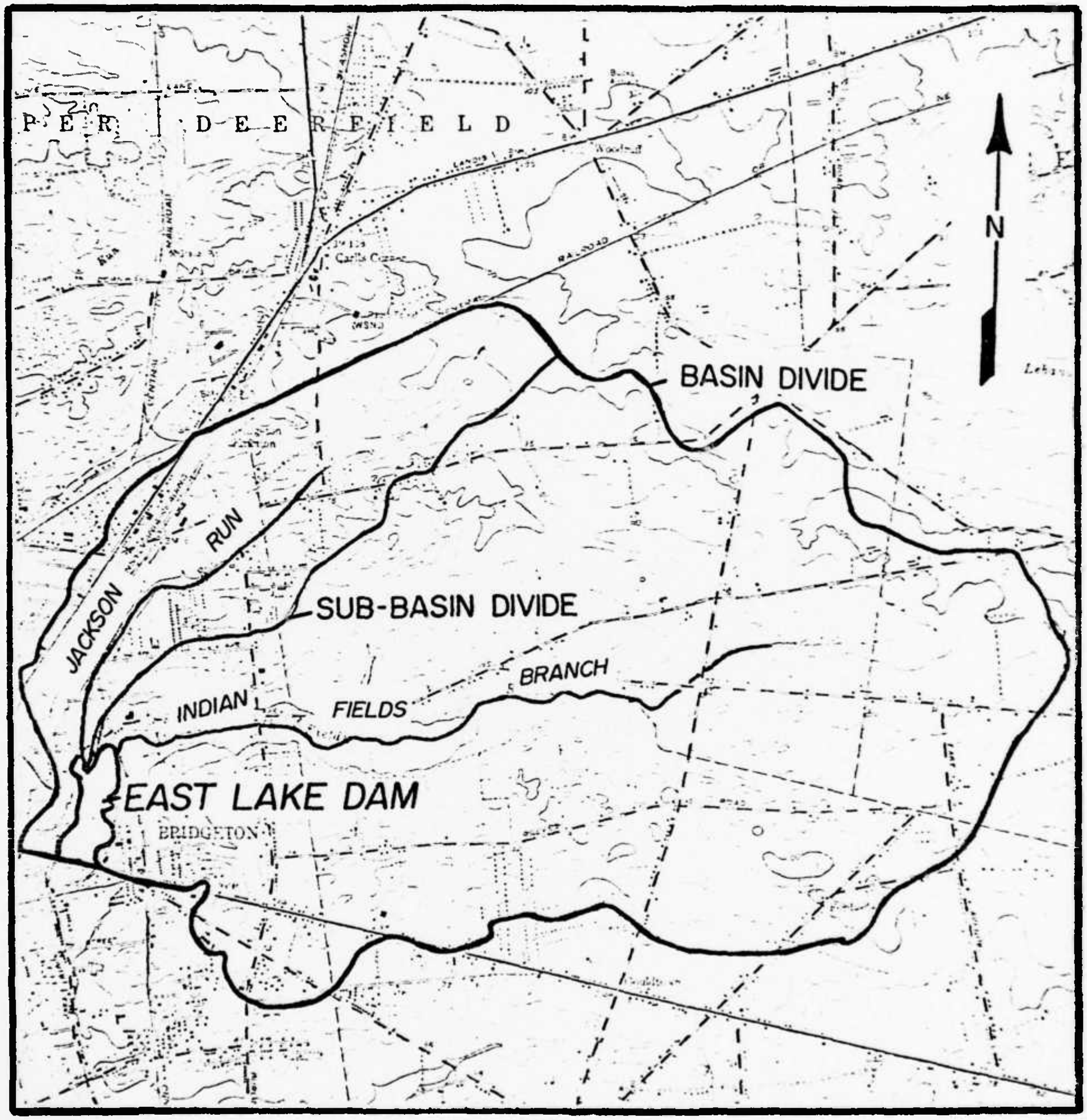


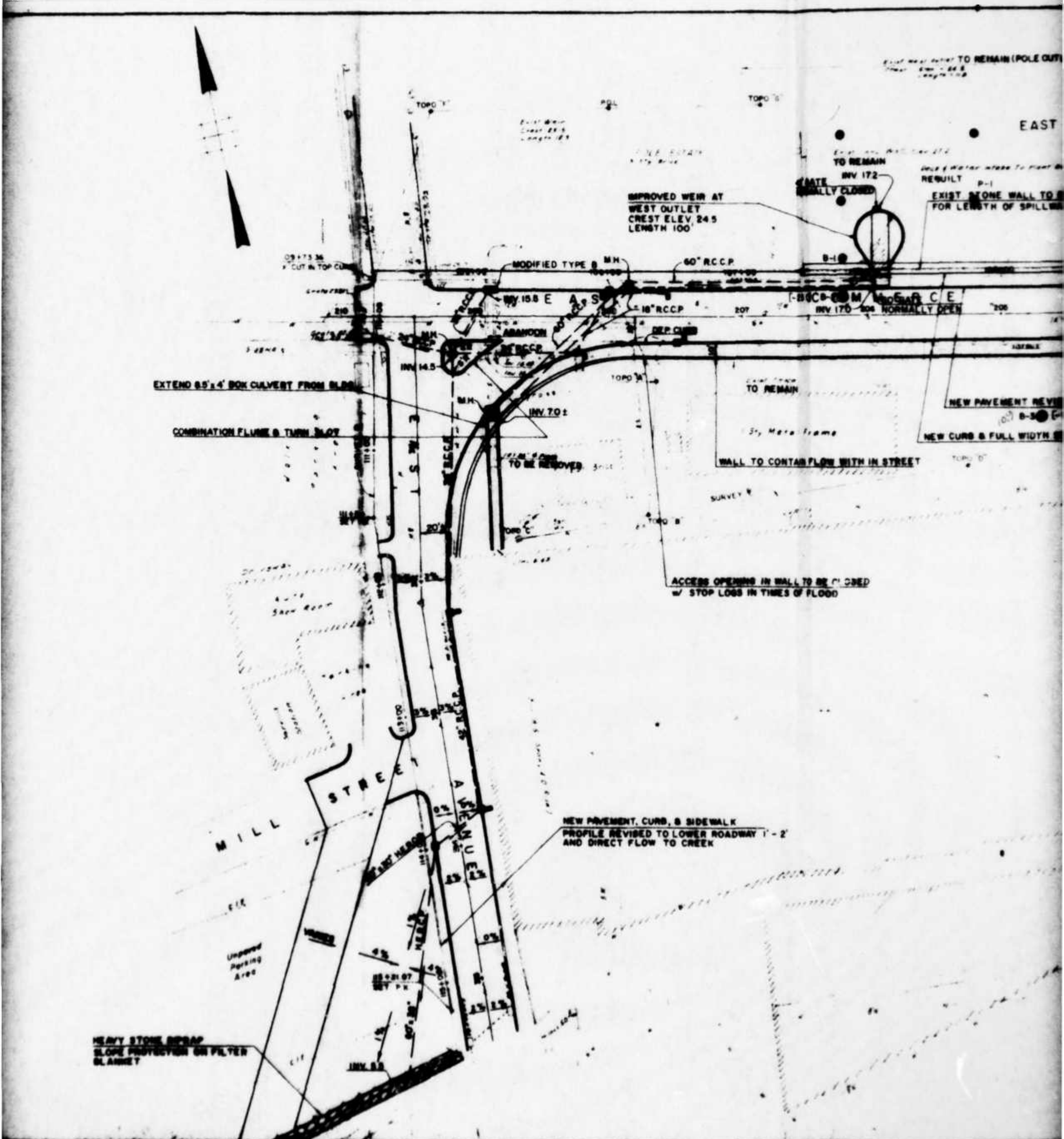
FIGURE I
REGIONAL VICINITY MAP
SCALE 1:24,000



SCALE 1" = 4,000'

FIGURE 2

DRAINAGE BASIN MAP


TO REMAIN (POLE OUT)
EAST
TO REMAIN
INV 17.2
REBUILT
P-1
IMPROVED WEIR AT
WEST OUTLET
CREST ELEV. 24.5
LENGTH 100'
EXIST STONE WALL TO
FOR LENGTH OF SPILLWA
MODIFIED TYPE B
M.H.
60" R.C.C.P.
INV 15.8
E A S
18" R.C.C.P.
INV 17.0
NORMALLY OPEN
ABANDON
INV 14.5
DEP CURB
EXTEND 8.5' x 4' BOX CULVERT FROM BLDG.
M.H.
INV 7.0 ±
TO REMAIN
NEW PAVEMENT REVI
COMBINATION FLUME & TURN SLOT
TO BE REMOVED
WALL TO CONTAIN FLOW WITH IN STREET
NEW CURB & FULL WIDTH
SURVEY
ACCESS OPENING IN WALL TO BE CLOSED
w/ STOP LOGS IN TIMES OF FLOOD
MILL STREET
AVENUE
NEW PAVEMENT, CURB, & SIDEWALK
PROFILE REVISED TO LOWER ROADWAY 1' - 2'
AND DIRECT FLOW TO CREEK
Unpaved
Parking
Area
HEAVY STONE RIPRAP
SLOPE PROTECTION ON FILTER
BLANKET
INV. 8.5

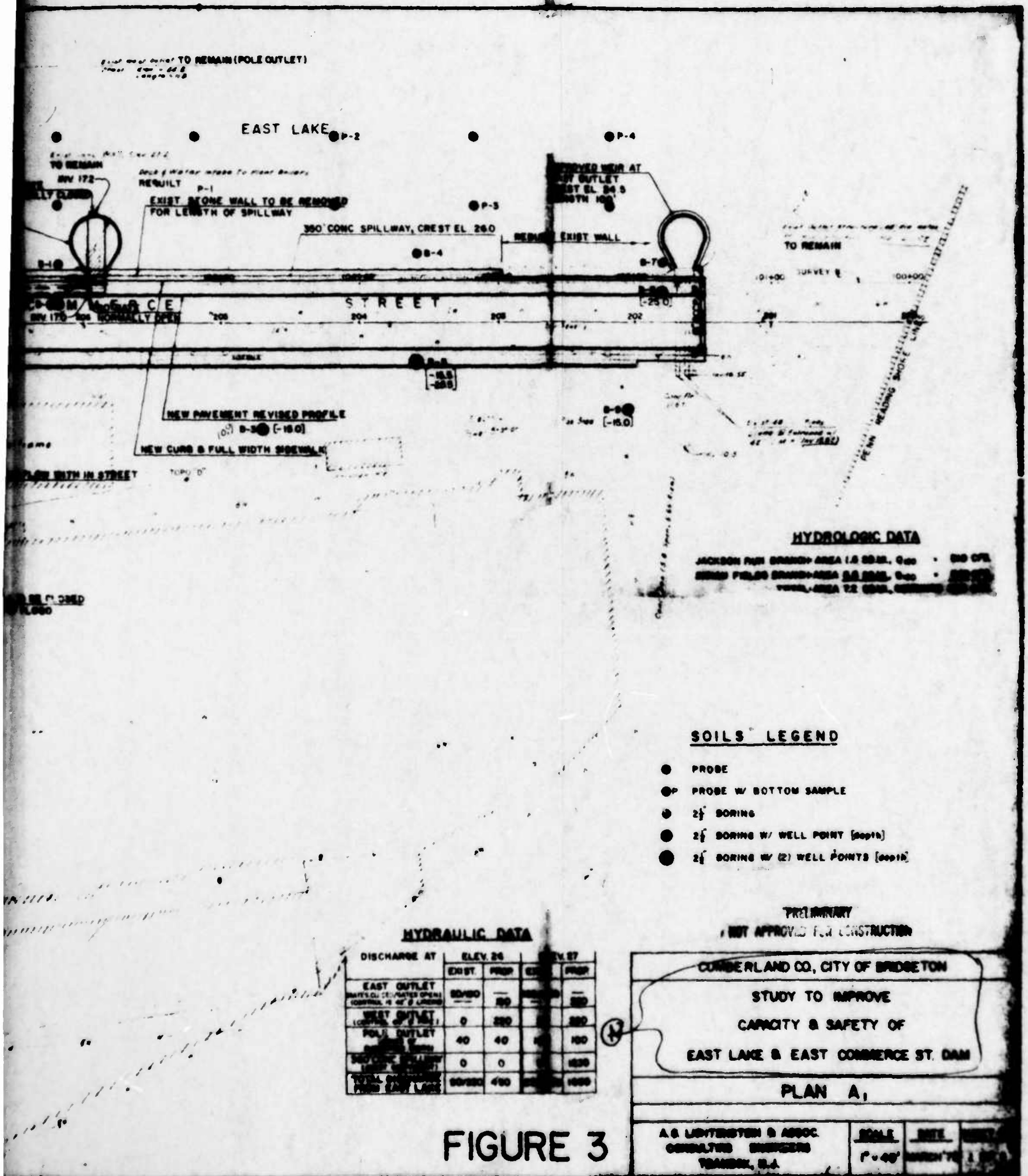
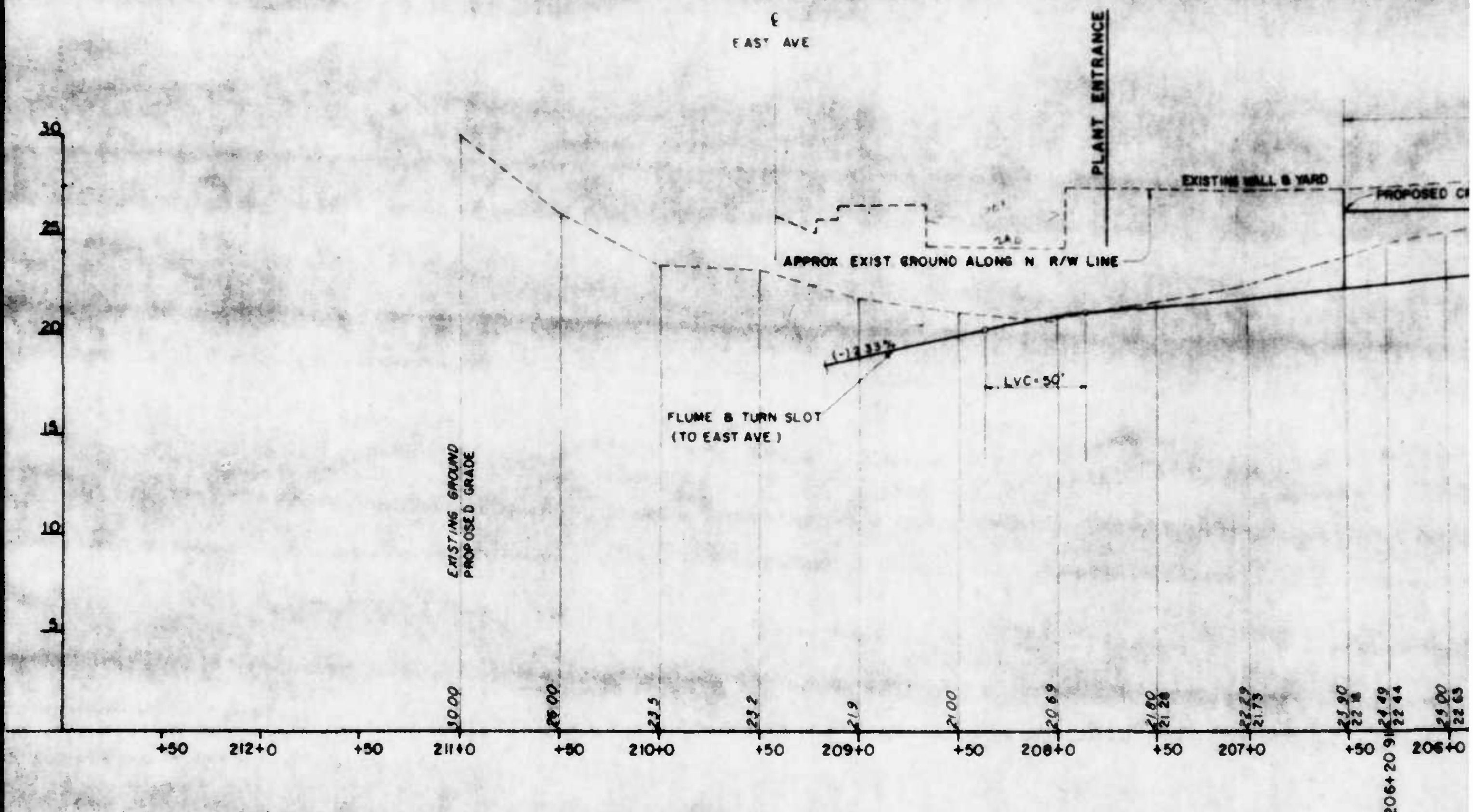

TO REMAIN (POLE OUTLET)
EAST LAKE
P-2
P-4
TO REMAIN
RESUILT
P-1
EXIST STONE WALL TO BE REMOVED FOR LENGTH OF SPILLWAY
P-3
350' CONC SPILLWAY, CREST EL 26.0
B-4
EXIST WALL
TO REMAIN
B-1
B-7
STREET
NEW PAVEMENT REVISED PROFILE
B-3 [-15.0]
B-6 [-15.0]
NEW CURB & FULL WIDTH SIDEWALK
PENN READING SHORE LINE
HYDROLOGIC DATA
SOILS LEGEND
PROBE
PROBE W/ BOTTOM SAMPLE
2½" BORING
2½" BORING W/ WELL POINT [depth]
2½" BORING W/ (2) WELL POINTS [depth]
PRELIMINARY
NOT APPROVED FOR CONSTRUCTION
HYDRAULIC DATA
DISCHARGE AT
ELEV. 26
EXIST.
PROP.
EAST OUTLET
WEST OUTLET
POLE OUTLET
TOTAL DISCHARGE FROM EAST LAKE
CUMBERLAND CO., CITY OF BRIDGETON
STUDY TO IMPROVE
CAPACITY & SAFETY OF
EAST LAKE & EAST COMMERCE ST. DAM
PLAN A,
A.G. LICHTENSTEIN & ASSOC.
CONSULTING ENGINEERS
SCALE
1" = 40'
DATE

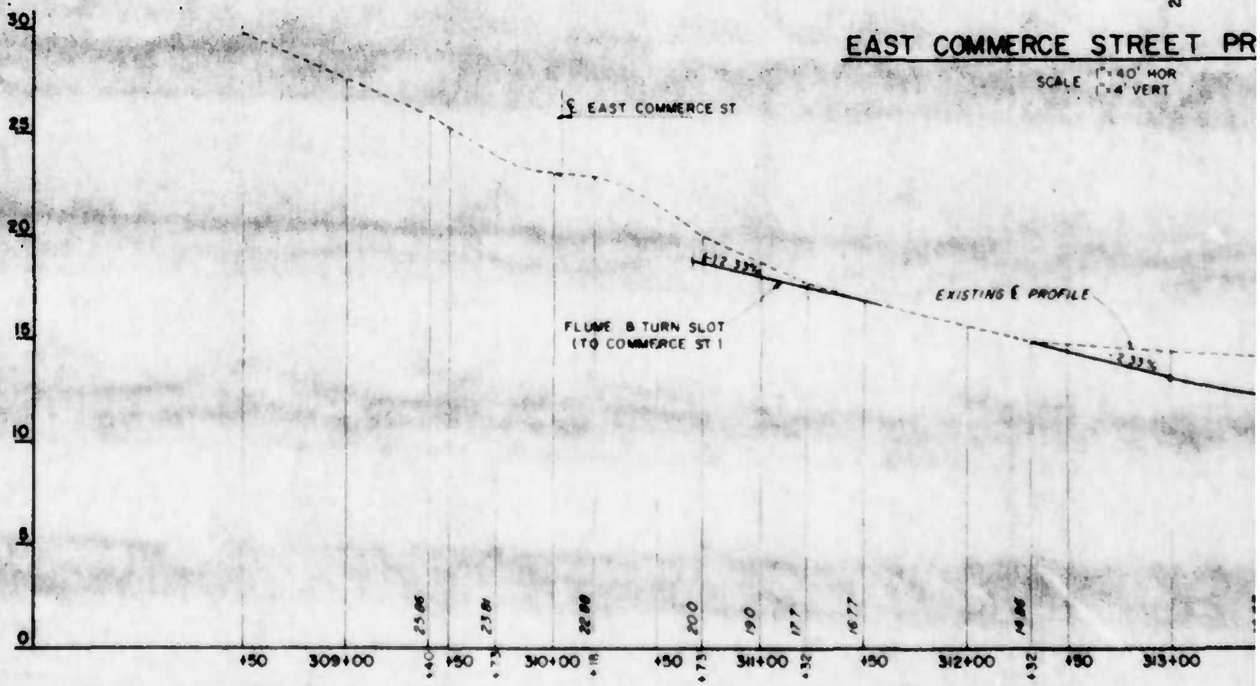
FIGURE 3

2



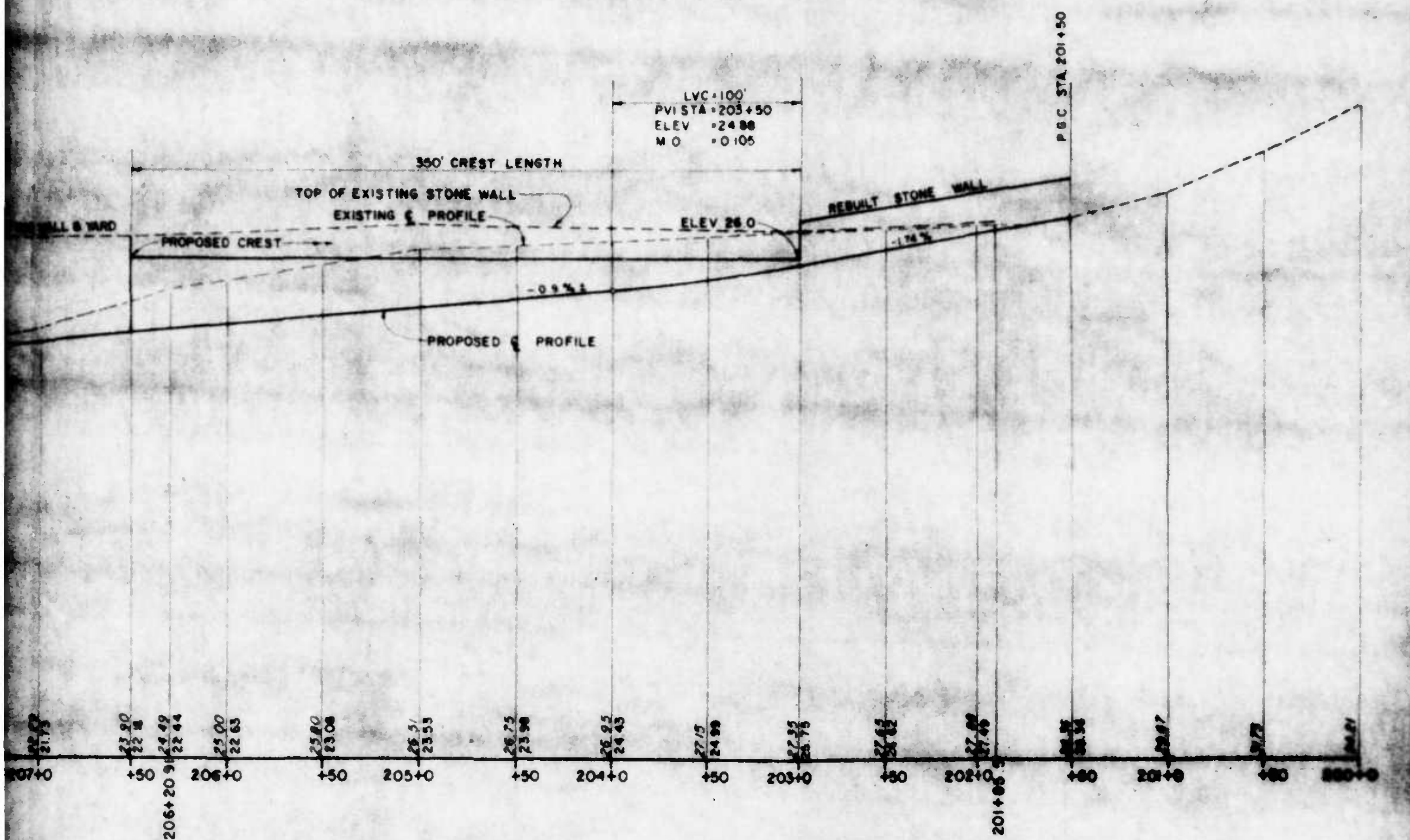
EAST COMMERCE STREET PR

SCALE 1" = 40' HOR, 1" = 4' VERT



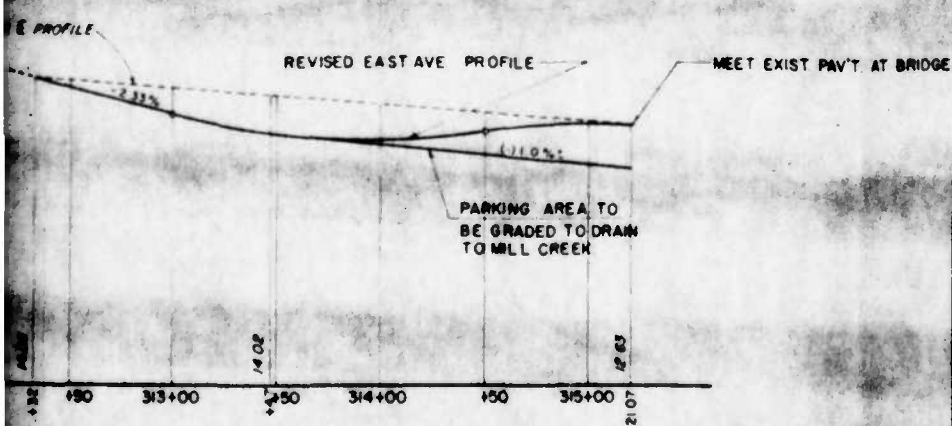
EAST AVENUE PROFILE

SCALE 1" = 40' HOR, 1" = 4' VERT



ERCE STREET PROFILE

SCALE 1"=40' HOR
1"=4' VERT



PRELIMINARY
NOT APPROVED FOR CONSTRUCTION

CUMBERLAND CO., CITY OF BRIDGETON

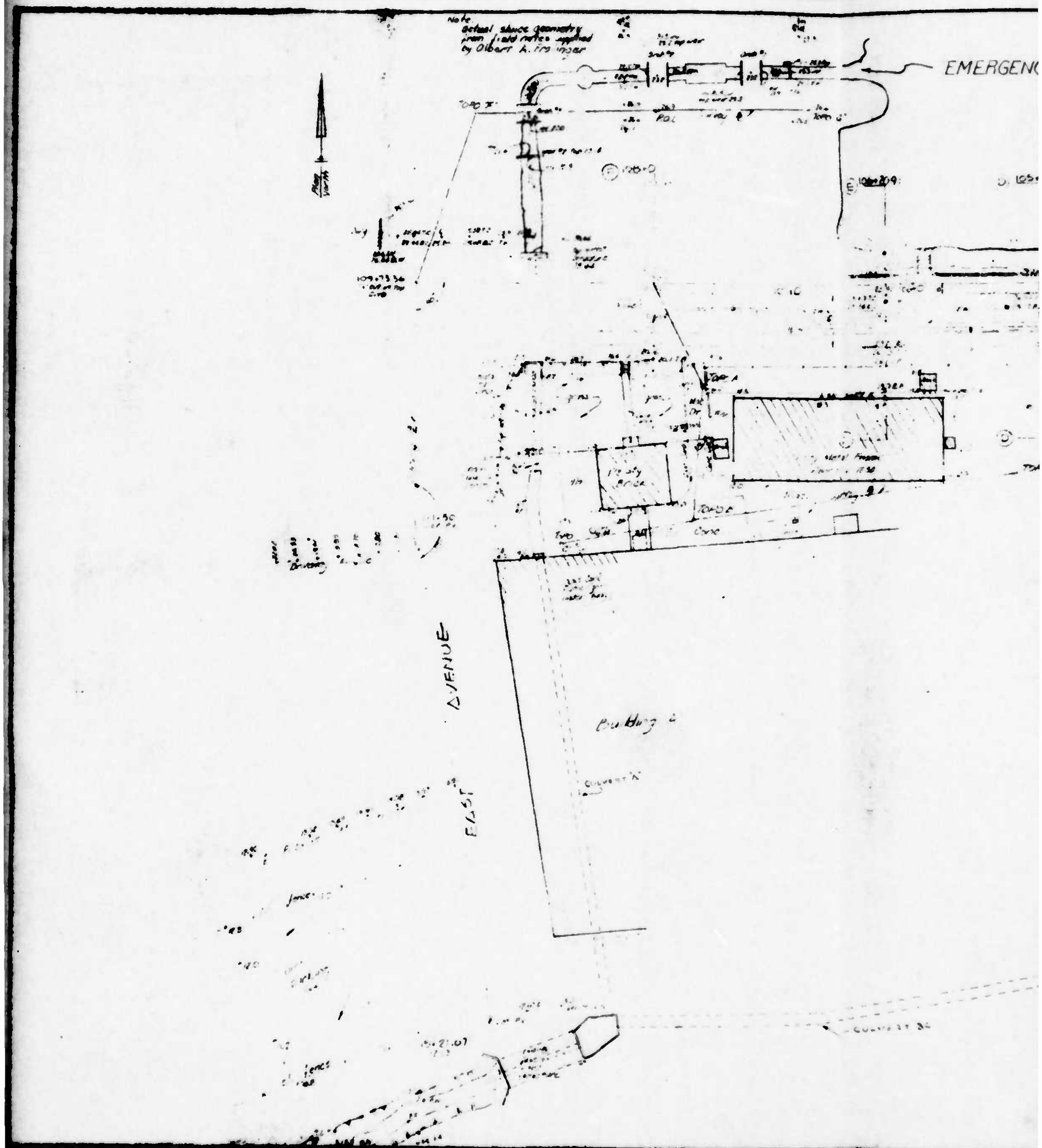
STUDY TO IMPROVE
CAPACITY & SAFETY OF
EAST LAKE & EAST COMMERCE ST. DAM

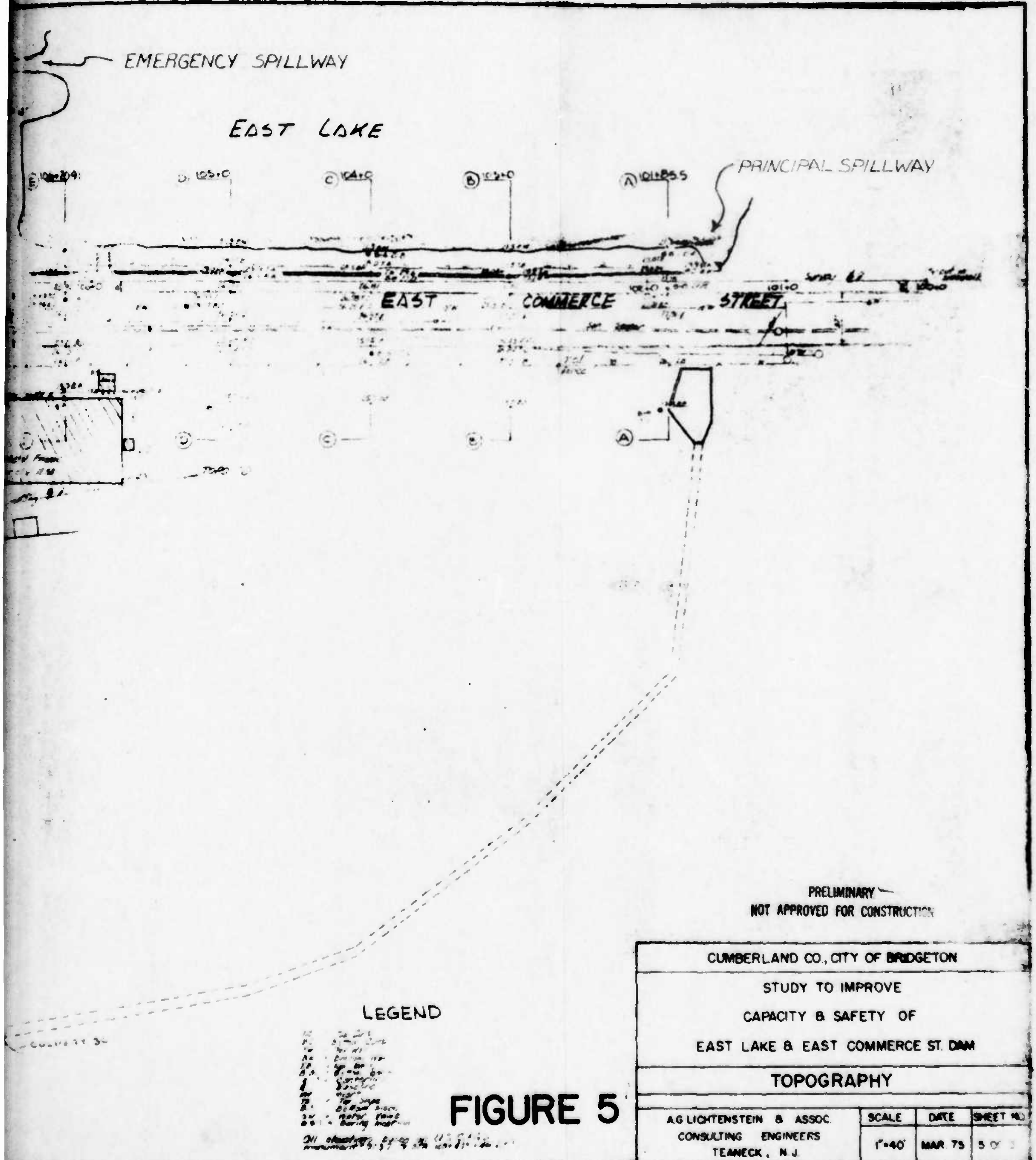
PROFILE

A.G. LICHTENSTEIN & ASSOC
CONSULTING ENGINEERS
TEANECK, N.J.

SCALE	DATE	SHEET
AS SHOWN	MAR '78	3 OF 8

FIGURE 4

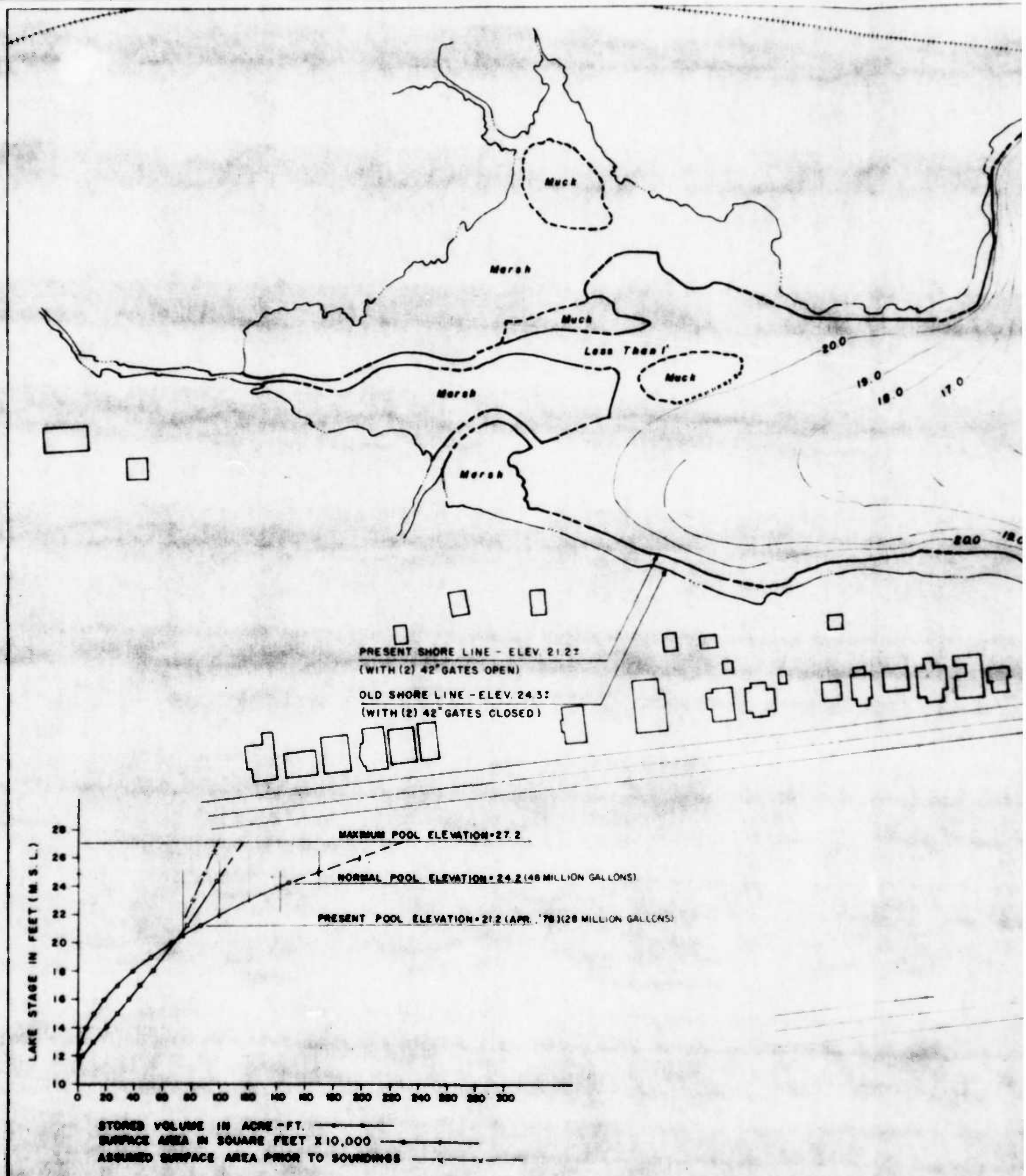

EMERGENC
AVENUE
EAST
Building



PRELIMINARY
NOT APPROVED FOR CONSTRUCTION

CUMBERLAND CO., CITY OF BRIDGETON			
STUDY TO IMPROVE CAPACITY & SAFETY OF EAST LAKE & EAST COMMERCE ST. DAM			
TOPOGRAPHY			
A.G. LICHTENSTEIN & ASSOC. CONSULTING ENGINEERS TEANECK, N.J.	SCALE: 1"=40'	DATE: MAR. 75	SHEET NO.: 5 OF [illegible]

FIGURE 5



Muck
Marsh
Muck
Less Than 1'
Muck
Marsh
Marsh
20.0
19.0
18.0
17.0
20.0
PRESENT SHORE LINE - ELEV. 21.2±
(WITH (2) 42" GATES OPEN)
OLD SHORE LINE - ELEV. 24.3±
(WITH (2) 42" GATES CLOSED)
MAXIMUM POOL ELEVATION = 27.2
NORMAL POOL ELEVATION = 24.2 (48 MILLION GALLONS)
PRESENT POOL ELEVATION = 21.2 (APR. '78)(28 MILLION GALLONS)
LAKE STAGE IN FEET (M.S.L.)
STORED VOLUME IN ACRE-FT.
SURFACE AREA IN SQUARE FEET X 10,000
ASSUMED SURFACE AREA PRIOR TO SOUNDINGS

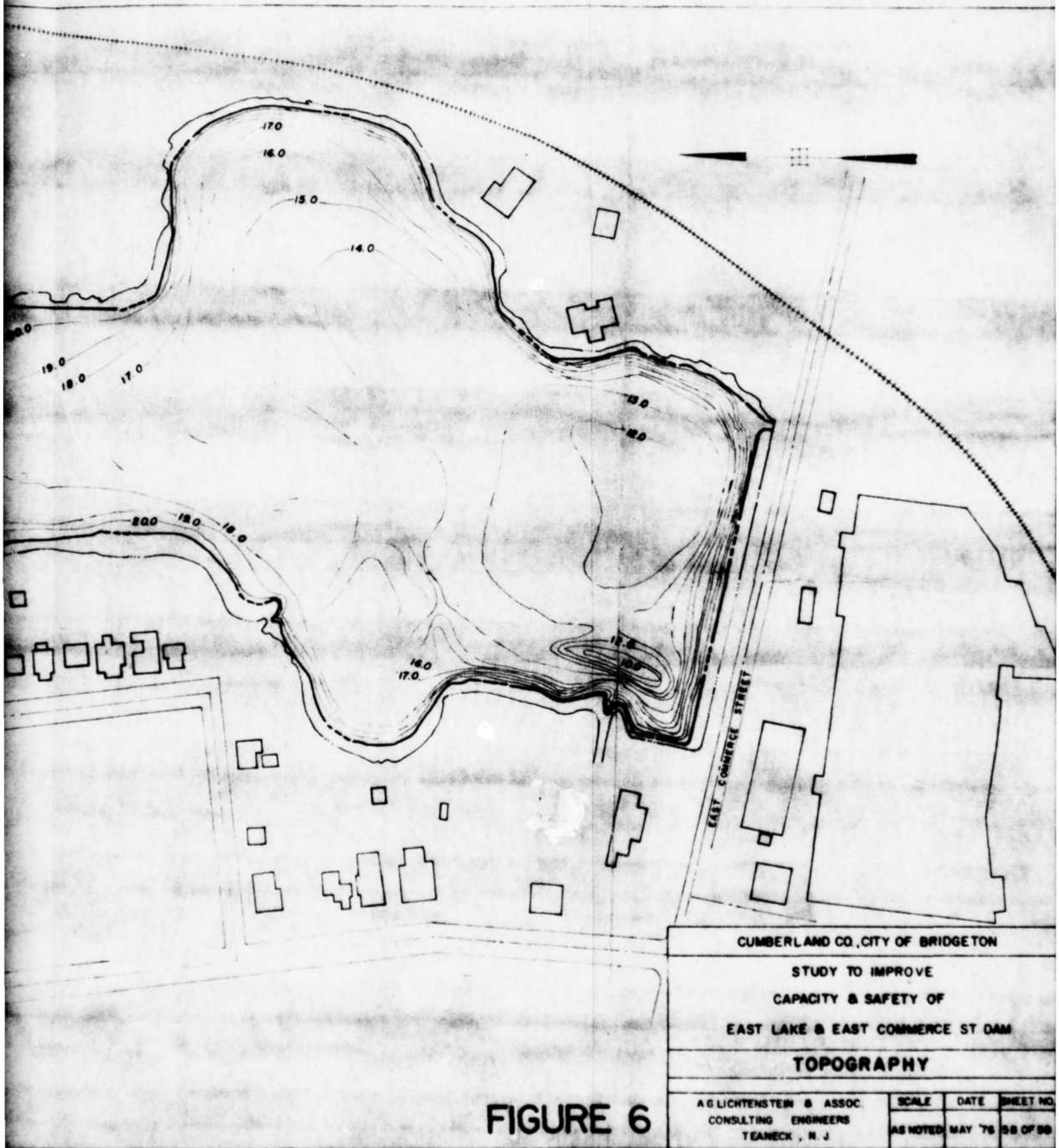
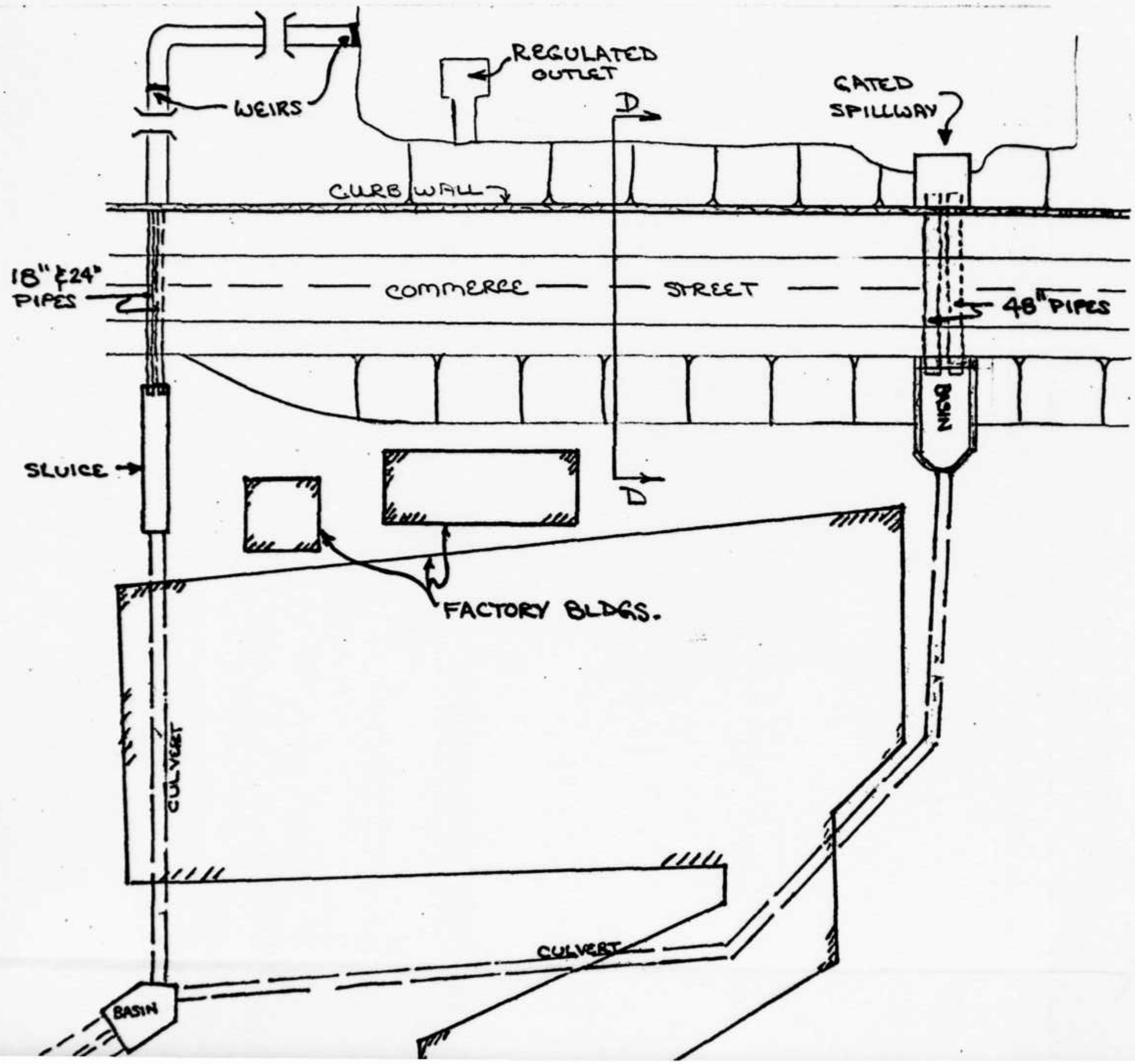
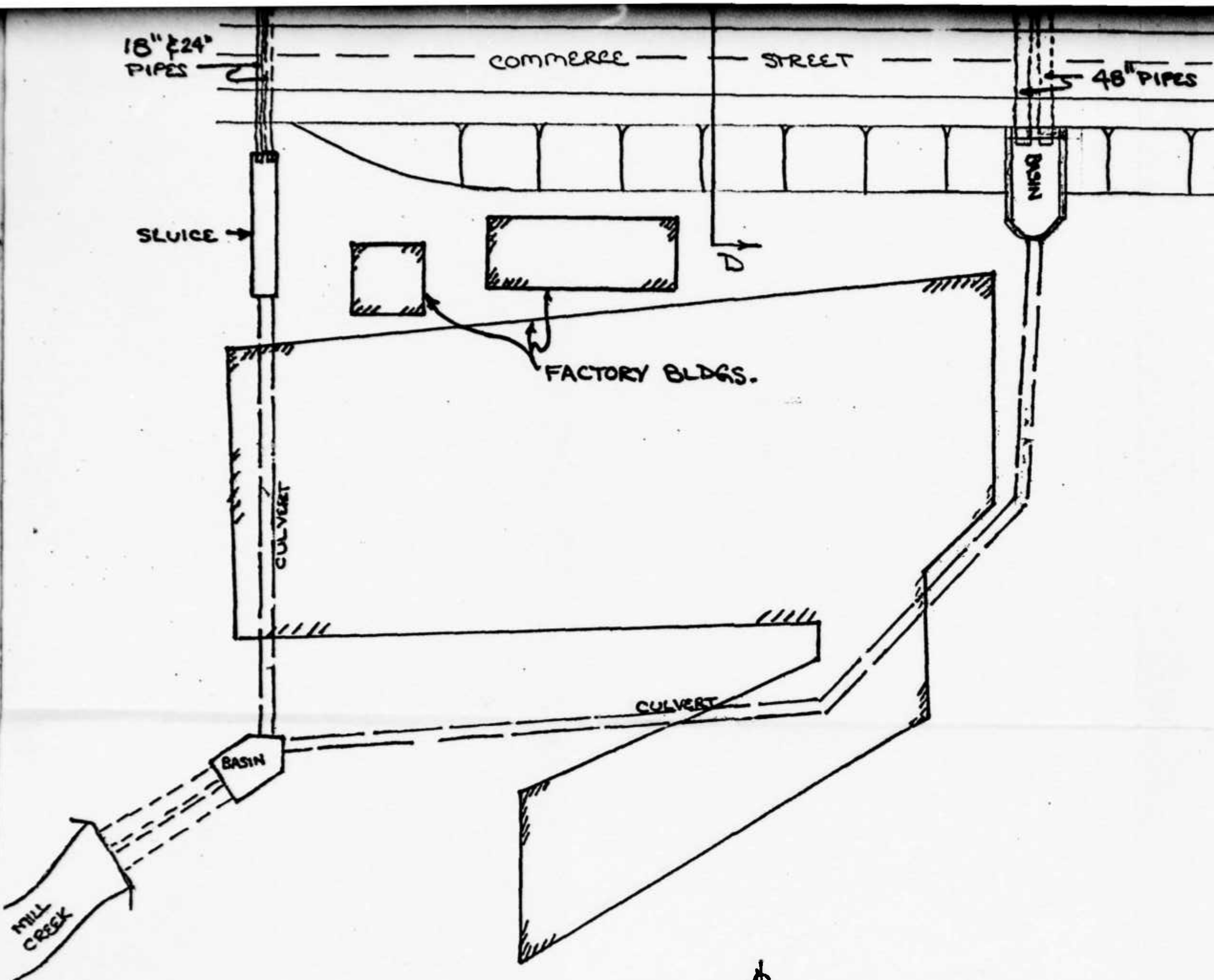
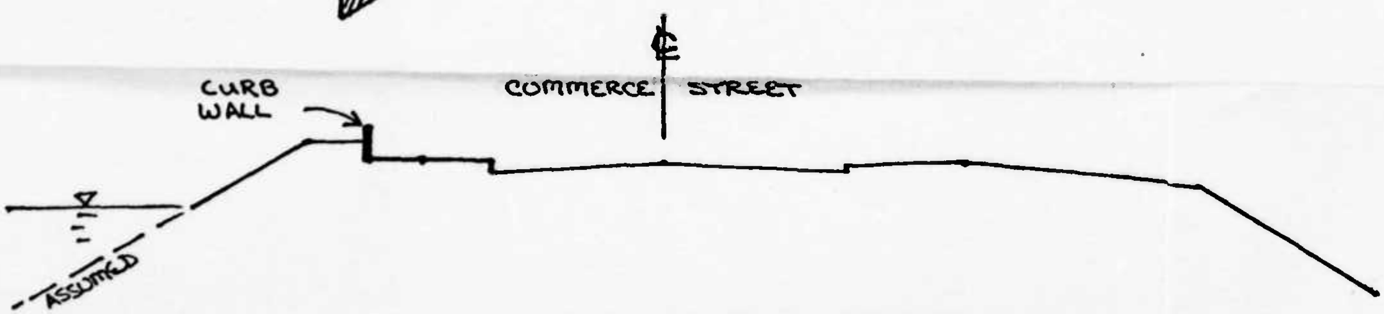

17.0
16.0
15.0
14.0
19.0
18.0
17.0
13.0
12.0
20.0
16.0
17.0
EAST COMMERCE STREET
CUMBERLAND CO., CITY OF BRIDGETON
STUDY TO IMPROVE
CAPACITY & SAFETY OF
EAST LAKE & EAST COMMERCE ST. DAM
TOPOGRAPHY
A.G. LICHTENSTEIN & ASSOC.
CONSULTING ENGINEERS
TEANECK, N.J.
SCALE
AS NOTED
DATE
MAY '76
SHEET NO.

FIGURE 6


REGULATED OUTLET
WEIRS
GATED SPILLWAY
D
CURB WALL
18" & 24" PIPES
COMMERCE STREET
48" PIPES
BASIN
SLUICE
D
FACTORY BLDGS.
CULVERT
CULVERT
BASIN





SECTION D-D FROM FIGURES

FIGURE 6A

2

THOR ENGINEERS 341 SYLVAN AVENUE LEONIA, NEW JERSEY 07605 201-944-5958	CLIENT PROJECT East Commerce Street Dam Study LOCATION Bridgeton, NJ NUMBER #7426	HOLE NO. [illegible] SHEET 1 OF 1 TYPE VERT. CASED DATE 11-14-74

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR	
At 10 ft. after 0 Hours	TYPE	Flush	SS		SURFACE ELEV. +22.3
	SIZE I.D.	3-1/2	1-3/8		DATUM USC & GS
At - ft. after - Hours	Hammer Wt.	300	148	BIT	BORING CONTR. Craig Testing
	Hammer Fall	18	30		BORING FOREMAN M. Pratt
					INSPECTOR L. Markowski

LOCATION:
West abutment at upstrean tow of embankment (waters edge)

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY 18	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
	1	0-	SS1	1	0	0	1	(+22.3)	Tan br. very loose cmf SAND, tr. f gravel, silt	
	1	2								
	0	2-	SS2	1	0	1	1	4.0		
	1	4								
5	1	4-	SS3	1	1	0	1		Br. very loose cmf SAND, sm cmf gravel, tr silt	
	9	6						6.0		
	15	6-	SS4	2	6	13	14	(+16.3)	Br. very dense SAND, sm cmf gravel, tr silt	
	21	8								
	33	8-	SS5	13	19	24	29			
10	32	10								
	11	10-	SS6	10	11	14	6			
	9	12								
	12	12-	SS7	8	12	23	26			
	15	14								
15	20	14-	SS8	54	65	83		15.5		
								(+6.8)	Bottom of boring at 15.5 feet	
20										

34

FIGURE 7

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

CLIENT
PROJECT East Commerce Street Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B2
SHEET 1 OF 1
TYPE HSA
DATE 11/15/74

GROUND WATER OBSERVATION
At 1'0" ft after 24 Hours
At 12.8 ft. after 25 days

	CASING	SAMPLER	CORE BAR
TYPE	HSA	SS	
SIZE I.D.	4"	1-3/8	
Hammer Wt.		140	BIT
Hammer Fall		30	

SURFACE ELEV. +24.0
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION:
South of curb for crest roadway at west abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
	-	0-1		Drilled				1.0	4-1/2" Asphalt	
		1-3	SS1	3	3	9	12	(+23.0)		
		3-5	SS2	10	9	8	8		Bt tan med dense to dense cmf SAND & cmf GRAVEL, tr silt	
5		5-7	SS3	8	9	5	3			
		7-9	SS4	2	2	1	2			
10		9-11	SS5	15	22	14	12			
		11-13	SS6	10	20	14	13	13.0		
		13-15	SS7	10	7	6	5	15.0	Bryel med dense SAND, sm mf gravel, tr silt, org. m-	
15		15-17	SS8	6	10	16	29	(+9.0)		
		17-19	SS9	13	36	55	74		Br yel gy very dense cmf SAND, trf gravel, silt	
20								22.0		
								(+2.0)	Br gy very dense cmf SAND, sm mf gravel	
25								26.5		
								(-1.5)	Bottom of boring at 26.5 feet — —	Observation well installed at 25. feet
30										

FIGURE 8

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

CLIENT	
PROJECT	East Commerce Street Dam Study
LOCATION	Bridgeton, NJ
NUMBER	#7426
HOLE NO.	B3
SHEET	1 OF 1
TYPE	VERT. HSA
DATE	11/15/74

GROUND WATER OBSERVATION

At 1.5 ft. after 0 Hours

At 2.1 ft. after ______ Hours 25 days

	CASING	SAMPLER	CORE BAR
TYPE	HSA	SS	
SIZE I.D.	3-1/2"	1-3/8"	
Hammer Wt.		140	BIT
Hammer Fall		30	

SURFACE ELEV.	+13.0
DATUM	USC & GS
BORING CONTR.	Craig Testir
BORING FOREMAN	M. Pratt
INSPECTOR	L. Markowski

LOCATION: Downstream toe of slope - west abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	18–24 RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-2	S-1	4	9	12	13	(+13.0)	Br yel med dense cmf SAND, tr/f gravel, tr silt	
		2-4	S-2	4	5	8	9	3.5		
5		4-6	S-3	15	22	22	29	(+9.5)	Br very dense SAND& cmf GRAVEL, tr silt	
		6-8	S-4	28	39	50	64	8.0		
		8-10	S-5	8	18	23	37	(+5.0)	Br very dense cmf SAND, tr silt, tr gravel	
10										
15								15.0		
								(+2.0)	Bottom of boring at 15.0 feet	Observation well 15.0 feet

FIGURE 9

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

CLIENT
PROJECT East Commerce Street Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B-
SHEET 1 OF
TYPE VERT CA
DATE 11-13-

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR
At 0.5 ft. after 0 Hours	TYPE		SS	
At ___ ft. after ___ Hours	SIZE I.D.	2-1/2	1-3/8	
	Hammer Wt.	300#	140#	BIT
	Hammer Fall	24"	30"	

SURFACE ELEV. +21.8
DATUM USC & GS
BORING CONTR. Craig Tes
BORING FOREMAN M. Prat
INSPECTOR T. Otto

LOCATION: Middle of embankment - upstream toe of slope

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-2	1	1	0	0	0	(+21.8)	Tan br very loose cmf SAND, tr mf gravel, tr silt	
		2-4	2	1	0	0	1			
5		4-6	3	1	1	0	1	6.0		
		6-8	4	1	1	2	1	(+15.8)	Br very loose cmf SAND & mf GRAVEL, tr silt	
		8-10	5	1	1	1	6			
10		10-12	6	3	1	1	1	12.0		
		12-14	7	Pushed				13.5	Br bl gy PEAT & ORGANIC SILT (Fi	
15		14-16	8	1	3	7	16	(+8.3)	Br gy med dense cmf SAND & mf GRAVEL	
								18.5 (+3.3)		
20		20-22	9	14	12	21	12		Tan ye br dense cmf SAND, tr/f gravel, tr silt t	
								22.0 (-0.2)	Bottom of boring at 22.0 feet	
25										
30										
35										

FIGURE

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5958

PROJECT	East Commerce Street Dam Study
LOCATION	Bridgeton, NJ
NUMBER	#7426
HOLE NO.	[illegible]
SHEET	1 OF 1
TYPE	VERT HSA
DATE	11/13/74

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR
At 16.0 ft. after 0 Hours	TYPE	HSA	SS	
At 14.1 ft. after 25 days	SIZE I.D.	4"	1-3/8	
	Hammer Wt.		140#	BIT
	Hammer Fall		30"	

SURFACE ELEV. +25.5
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION: Middle of south sidewalk - middle of embankment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-1.3	S1A					(+25.5) 1.3	Concrete sidewalk slab and base	
		2-4	S1B	5	6	5	5	(+24.2)	Bl br ORG material, cinders	
								3.5	Bl br soft silty CLAY	
5		4-6	S2	5	6	6	6	(+21.5) 6.0	Br med dense mf SAND & CINDERS, org material	
		6-8	S3	4	5	3	2	(19.5) 8.0	Br. Bl. loose SAND, tr clay, tr silt, org matl	
		8-10	S4		Pushed			(17.5)	Bl br org mtl, ashes cinders	
10		10-12	S5	1	1	1	1	12.0		
		12-14	S6	2	3	3	4	(+13.5)	Br bl org mtl, ashes wood chips, sawdust	
15		14-16	S7A	5	28	17	11	15.0		
			S7B					16.0	Gy cmf SAND, wood	
		16-18	S8	10	12	8	3	(+9.5) 18.0	Gy br med dense cmf SAND, tr	
		18-20	S9	12	23	24	26	(+7.5)	Gy br dense cmf SAND, tr silt (org mtl interbedded)	
20								22.0		
								(+3.5)	Br very dense cmf SAND, sm to tr mf gravel	
25		25-26.5	S10	18	47	71				
30		30-31.5	S11	8	17	29				
35		35-35.5	S12	122				35.5 (-10.0)	Bottom of Boring at 35.5 feet	Well points installed at 15.5 feet and 25.5 feet depths.

FIGURE II

THOR ENGINEERS
341 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

CLIENT
PROJECT East Commerce St. Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B7
SHEET 1 OF 1
TYPE VERT. CASI
DATE 11-13-74

GROUND WATER OBSERVATION		CASING	SAMPLER	CORE BAR
At 2" ft after 0 Hours	TYPE	Flush	SS	
At ___ ft. after ___ Hours	SIZE I.D.	2-1/2"	1-3/8"	
	Hammer Wt.	300#	140#	BIT
	Hammer Fall	24"	30"	

SURFACE ELEV. 21.4
DATUM USC & GS
BORING CONTR. Craig Testi
BORING FOREMAN M. Pratt
INSPECTOR T. Otto

LOCATION:
Upstream toe of slope east abutment adjacent to spillway structure.

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18 (18-22)	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
0		0-2	SS1	2	3	1		(+21.4) 2.0	Br loose cmf SAND, sm mf gravel	
		2-4	SS2	2	1	1	1	(+19.4)	Dk br gy very loose cmf SAND, org mtl interbedded	
5		4-6	SS3	1	1	1	1			
		6-8	SS4	1	0	0	1			
		8-10	SS5	1	0	0	1	9.0	Dk gy soft cmf SAND & ORG SILT	
10		10-12	SS6	1	1	0	1	10.5	Br gy loose org silty cmf SAND	
		12-14	SS7	1	2	2	8	12.0	Dk br PEAT & ORG SILT	
								13.5	PEAT & WOOD fiber	
15		14-16	SS8	9	13	11	9	14.0 (+7.4)	By br med dense cmf SAND & cmf GRAVEL	
20								20		
		25-26	S10	5	9	17		(+1.4)	Tan ye br dense cmf SAND	
25								26.5 (-5.1)	Bottom of boring at 26.5 feet	
30										
35										

FIGURE 12

THOR ENGINEERS
741 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201-944-5958

PROJECT: East Commerce Street Dam Study
LOCATION: Bridgeton, NJ
NUMBER: #7426

HOLE NO. 22
SHEET 1 OF 1
TYPE VERT. HSA
DATE 11-15-74

GROUND WATER OBSERVATION
At 16.5 ft. after 24 Hours
At 14.1 ft. after 25 days Hours

	CASING	SAMPLER	CORE BAR
TYPE	HSA	SS	
SIZE I.D.	4"	1-3/8	
Hammer Wt.		140#	BIT
Hammer Fall		30"	

SURFACE ELEV. +27.7
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION: North curb for crest roadway at east abutment

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0–6	6–12	12–18	RECOVERY 18 >2	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
		0-2	S1	8	9	5	8	(+27.7) 2.0	Bl br dense cmf SAND & GRAVEL, cinders, wood, brick	
		2-4	S2	12	9	5	4	(+25.7)	Tan bn loose cmf SAND, tr silt tr clay, wood fibers	
5		4-6	S3	2	2	3	3	6.0		
		6-8	S4	7	3	4	4	(+21.7)	Tan loose cmf SAND, tr silt, clay f gravel	
		8-10	S5	7	4	3	3	10.0		
10		10-12	S6	4	3	3	2	(+17.7)	Br tan loose cmf SAND, tr silt, clay, f gravel	
		12-14	S7	4	2	2	2			
15		14-16	S8	3	2	2	3			
		16-18	S9	2	1	3	5	18.0		
		18-20	S10	3	3	9	21	(+9.7)	Gy br dense cmf gravelly SAND	
20		20-22	S11	25	43	37	27	23.0		
								(+4.7)	By br very dense cmf SAND, sm mf gravel	
25		25-26.5	S12	37	56	69		26.5		
								(+1.2)	Bottom of boring at 26.5 feet	Observation well installed at 25.0 feet
30										
35										

GROUND SURFACE TO [illegible] FT. USED HSA [illegible]

FIGURE 13

THOR ENGINEERS
541 SYLVAN AVENUE
LEONIA, NEW JERSEY 07605
201.944.5938

CLIENT
PROJECT East Commerce Street Dam Study
LOCATION Bridgeton, NJ
NUMBER #7426

HOLE NO. B9
SHEET 1 OF 1
TYPE VERT, HSA
DATE 11/15/74

GROUND WATER OBSERVATION
At - ft. after - Hours
At 3.5 ft. after 25 days Hours

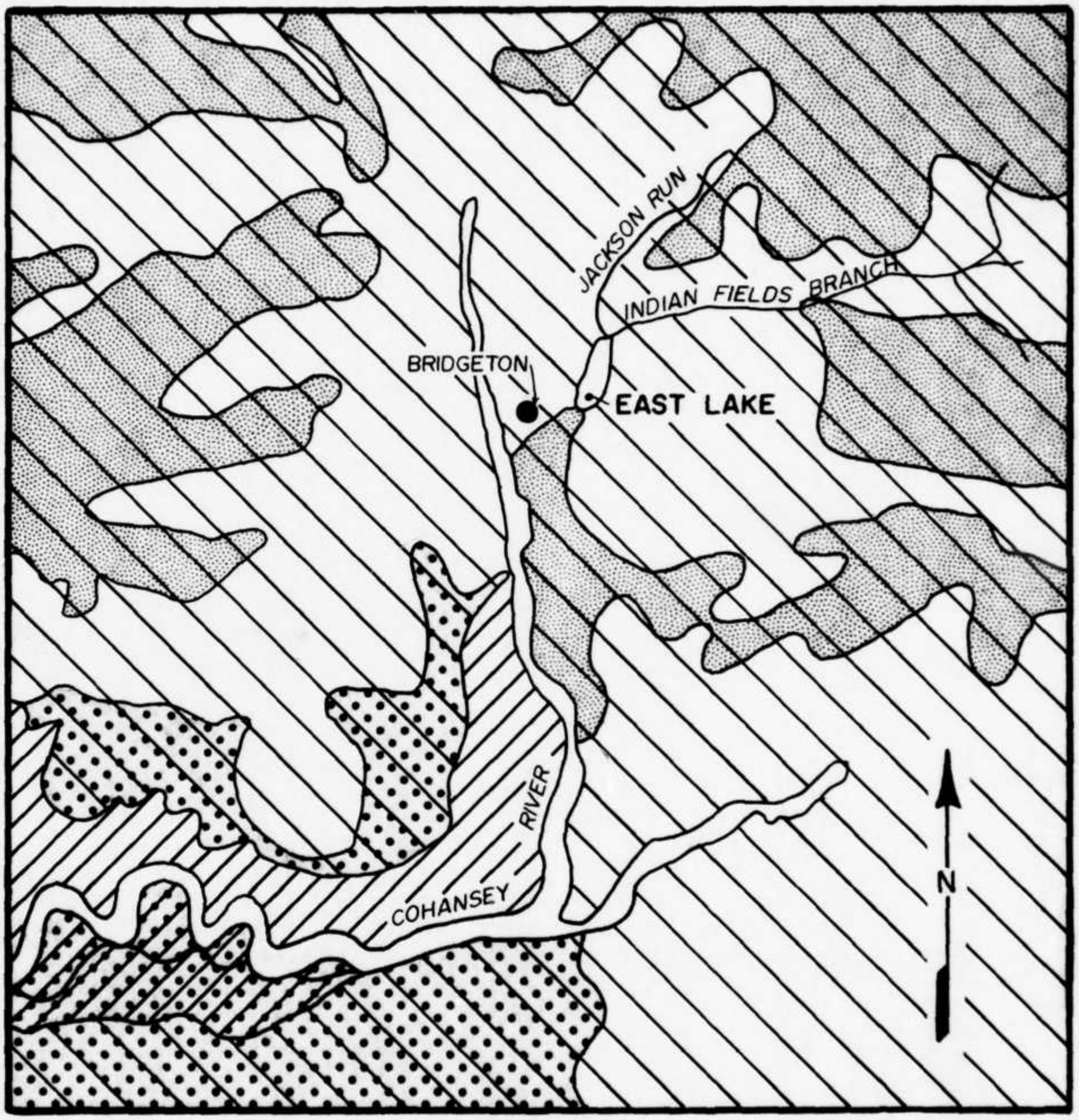
	CASING	SAMPLER	CORE BAR
TYPE	HSA	SS	
SIZE I.D.	4"	1-3/8"	
Hammer Wt.		140#	BIT
Hammer Fall		30#	

SURFACE ELEV. +17.7
DATUM USC & GS
BORING CONTR. Craig Testing
BORING FOREMAN M. Pratt
INSPECTOR L. Markowski

LOCATION: Downstream toe of embankment adjacent to spillway and east abutment.

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE DEPTHS from/ to	NO. & TYPE of Sample	BLOWS PER 6" ON SAMPLER 0—6	6—12	12—18	18-24	RECOVERY	STRATA CHANGE DEPTH Elev.( )	FIELD DESCRIPTION	REMARKS
		0-2	S1	6	8	8		9	(+17.7) 2.0	Bl br med dense cmf SAND & cmf GRAVEL, cinders	
		2-4	S2A S2B	6	8	6		12	3.5	Tan br med dense cmf SAND, sm mf gravel tr clay, peat	
5		4-6	3	7	6	4		7	(+14.2) 6.0	Tan br med dense cmf SAND, tr mf gravel, silt, clay	
		6-8	4	1	2	10		11	8.0	Br dense cmf SAND, sm mf gravel, tr clay	
10		8-10	5	8	12	11		13	(+9.7)	Br tan dense cmf SAND, tr mf gravel	
15									15.0 (+2.7)	Bottom of boring at 15.0 feet	Well observation installed at 15.0 feet
20											
25											

FIGURE 14



SCALE 1" = 4 MILES

LEGEND:

▧	COHANSEY SAND	QUARTZ SAND.
▨	KIRKWOOD SAND	FINE MICACEOUS SAND.
░	BRIDGETON FORMATION	GRAVEL AND SAND.
⁘	CAPE MAY FORMATION	GRAVEL AND SAND.

FIGURE 15
GEOLOGIC MAP

APPENDIX

FIELD INSPECTION REPORT

Check List
Visual Inspection
Phase 1

Name Dam East Lake Dam **County** Cumberland **State** New Jersey **Coordinators** Mr. Larry Woscyn. New Jersey DEP

Date(s) Inspection 2/3/78 **Weather** Clear **Temperature** 30's

Pool Elevation at Time of Inspection 171 **M.S.L.** **Tailwater at Time of Inspection** --- **M.S.L.**

Inspection Personnel:

Mr. Lee DeHeer   Mr. Gurbaksh Sanghera

Mr. George Elias

Mr. Stefan Manea

Mr. Gurbaksh Sanghera **Recorder**

Accompanied by:

Mr. Larry Woscyna, New Jersey Department of Enviromental Protection

Mr. Robert Plummer, Plant Engineer, Murbeck Fabrics

Mr. Burt Diamond, Plant Manager, Murbeck Fabrics

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	NONE NOTED	NONE
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	NONE NOTED	NONE
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	At several locations, bank sloughing has created vertical scarps of 2-3 feet on the upstream face. Settlement across the top of dam was noted at several locations with one settlement area of about 120 sq.feet with a maximum settlement of 2.5 feet.	The upstream face should be repaired with a suitable material and provided with an appropriate riprap surface.
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	The stone and masonry wall across the top of the dam is presently misaligned, both horizontally and vertically, revealing differential movement of the embankment.	
RIPRAP FAILURES	Upstream face is not riprapped.	Face should be covered with heavy stone to protect the embankment.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Embankment erosion was noted to depths of 3-4 feet on both sides of the east outlet structure for a length of 30-50 feet.	These areas should be replaced with suitable materials and protected by training walls or heavy riprap.
ANY NOTICEABLE SEEPAGE	Seepage occurs along portions of the downstream face following periods of rainfall excess. The seepage appears to be due to inadequate storm drainage causing ponding along East Commerce St. and seems to be related to the settle-areas noted on the previous page.	Seepage might be controlled or eliminated through installation of adequate storm drainage facilities along the road atop the dam.
STAFF GAGE AND RECORDER	No gages are in use at this site.	NONE
DRAINS	French drains were installed at the toe of the embankment in 1955. Water accumulating in the drains is pumped out. According to Bob Plummer, Junior Plant Engineer at the Bridgeton Dyeing and Finishing Co., the drains maintain a ground water level about 6-inches below the floor of the plant.	NONE

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	The outlet conduit consists of two 48-inch cast iron pipes lined at their entrances with 42-inch corrugated metal pipes. The pipes are buried and no cracking or spalling was noted at exposed locations.	NONE
INTAKE STRUCTURE	The east outlet intake structure is badly deteriorated, with spalling at a number of locations. The columns are cracked and a large crack is located near the junction of the beam with the slab.	The intake appears to be deficient both structurally and hydraulically. Repair or replacement should be effected as merited.
OUTLET STRUCTURE	The two 48-inch outlet conduits are supported by a stone and masonry wall at the downstream face of the dam. Discharge falls into a concrete basin located about 5-feet below the pipe inverts.	No problems were noted.
OUTLET CHANNEL	The outlet channel is a box culvert 5.8 feet wide by 5.66 feet high carrying discharge about 900-feet from the concrete basin to Mill Creek.	No problems were noted.
EMERGENCY GATE	The intake structure is equipped with 48-inch circular gates. The gates are badly rusted and due to the questionable safety of this project, New Jersey DEP, in 1973, ordered that the gates be kept open at all times.	See remarks under "Intake Structure"

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Flat slopes, no signs of sliding	NONE
SEDIMENTATION	The reservoir appears to be moderately silted.	Siltation does not affect the stability of the dam.

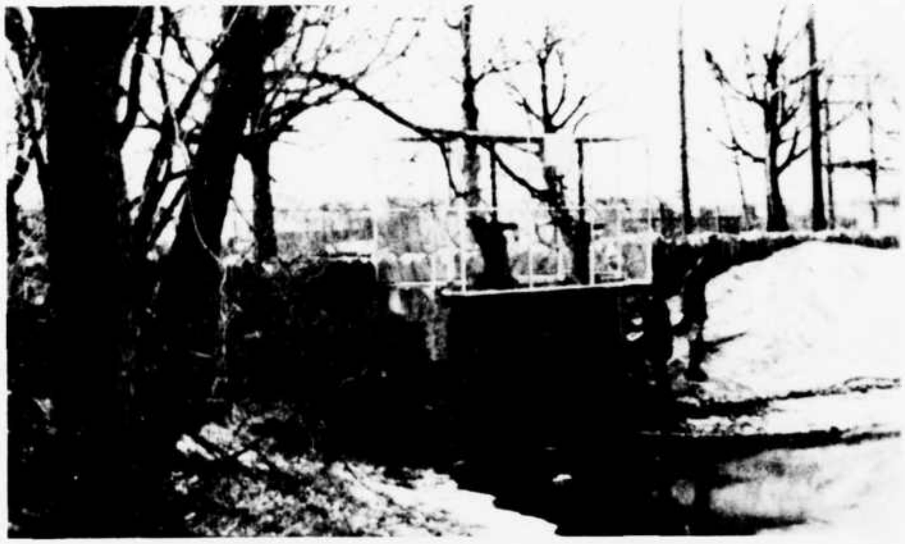
DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Flow from the dam to the down stream channel must travel through various open channels and culverts. The stream is subject to tidal variations that affect the discharge capacity of the outlet works.	NONE
SLOPES	No problems were observed.	NONE
APPROXIMATE NO. OF HOMES AND POPULATION	The area downstream of the dam consists of business and industry with some residential areas (approximately 1,000 people).	

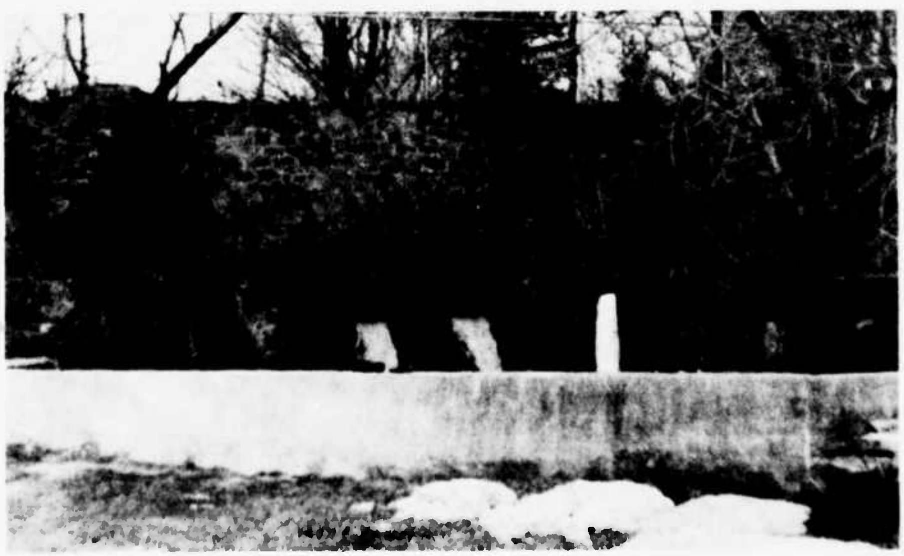
ITEM	REMARKS
MONITORING SYSTEMS	NONE
MODIFICATIONS	NONE
HIGH POOL RECORDS	No official records for this site. Unofficial information on flooding during the tropical storm of 1972 is included in Section 1.3.b.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	A previous study, including evaluation and proposed improvements to the dam outlet works, was completed by A.G. Lichtenstein and Associates, Consulting Engineers, Teaneck, NJ (dated August, 1976).
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	No information available.
MAINTENANCE OPERATION RECORDS	No records available.

ITEM	REMARKS
DESIGN REPORTS	The dam is reported to have been built in 1805. No records available.
GEOLOGY REPORTS	No information available.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	½ PMF - Inflow peak 12,500 cfs; routed outflow peak 12,450 cfs (29.9 feet above MSL). This discharge overtops the dam. The embankment is in poor condition. No seepage studies were made.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	Logs of eight soil borings (November, 1974) along the embankment are shown on Figures 7 through 14. The boring locations are shown on Figure 2.
POST-CONSTRUCTION SURVEYS OF DAM	See Figures 5 and 6
BORROW SOURCES.	Unknown.

PHOTOGRAPHS

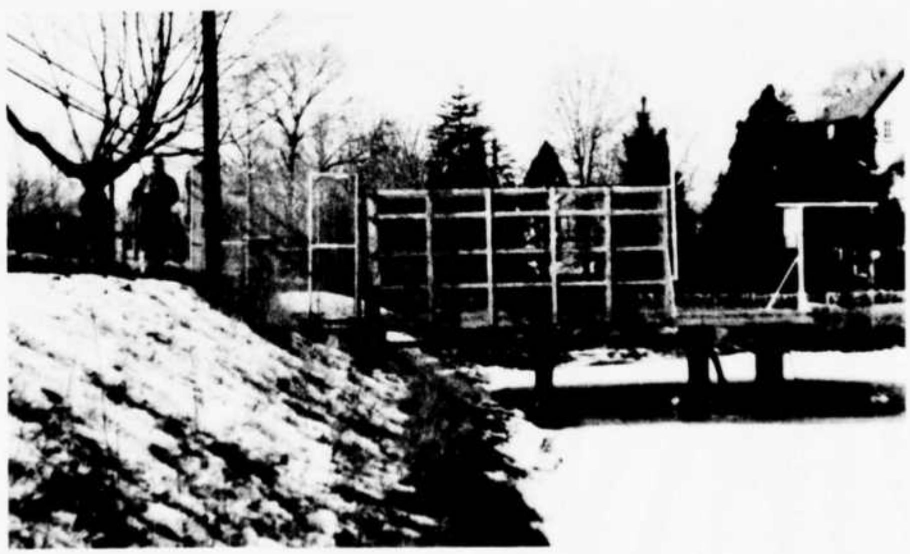


RESERVOIR OUTLET STRUCTURE AT EAST END OF EMBANKMENT



OUTFALL FROM RESERVOIR OUTLET STRUCTURE

BOX CULVERT DOWNSTREAM FROM OUTFALL



INTAKE STRUCTURE FOR INDUSTRIAL PLANT

SPILLWAY ON WEST SIDE OF RESERVOIR

SPILLWAY CHANNEL LOOKING DOWNSTREAM

TOP OF DAM LOOKING EAST—DOWNSTREAM SIDE

DOWNSTREAM FACE OF EMBANKMENT LOOKING WEST

HYDRAULIC AND HYDROLOGIC

CALCULATIONS

1800.001.

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PROBABLE MAXIMUM FLOOD CALCULATION

DRAINAGE AREAS

JACKSON RUN BRANCH - 1.36 SQ.MI.
INDIAN FIELDS BRANCH - 5.90 SQ.MI.

7.26 SQ.MI.

FROM HYDROMETEOROLOGICAL REPORT #33

6 HOUR - 10 SQ.MI. PMP = 27" ZONE #6

SINCE THE DRAINAGE AREA IS UNDER 10 SQ.MI., NO REDUCTION REFLECTING BASIN SIZE IS USED.

6 HR. PMP	1.00 × 27" =	27.0"
12 HR. PMP	1.09 × 27" =	29.4"
24 HR. PMP	1.17 × 27" =	31.6"
48 HR. PMP	1.26 × 27" =	34.0"

PMP VALUES ARE REDUCED 20% FOR PROBABLE MISALIGNMENT OF BASIN & STORM ISOHYETALS.

6 HR PMP	- 21.6"		
12 HR PMP	- 23.5"	(23.5-21.6)/6 = 1.9/6 = .32 "/HR	*
24 HR PMP	- 25.3"	(25.3-23.5)/6 = 1.8/12 = .15 "/HR	*
48 HR PMP	- 27.2"	(27.2-25.3)/6 = 1.9/24 = .08 "/HR	*

* THESE PRECIPITATION INCREMENTS CAN BE NEGLECTED.

ESTIMATION OF T_c

BUREAU OF PUBLIC ROADS

JACKSON RUN BRANCH

$T_c = (11.9\,L^3/H)^{.385}$ L: 2.69 MILES H = 110' - 25' = 85'

T_c = 1.5 HOURS

INDIAN FIELDS BRANCH

$T_c = (11.9\,L^3/H)^{.385}$ L = 4.02 MILES H = 110' - 25' = 85'

T_c = 2.34 HOURS

UPLAND METHOD (SCS)

JACKSON RUN BRANCH

①

AVERAGE LAND SLOPE ≃ 2.0% UPLAND LENGTH (L_u) = 70

VELOCITY = 1.5 fps (NEARLY BARE GROUND)

T_{t_1} = 7000 / 1.5 = 4667 sec = 78 min = 1.3 hours

②

STREAM LENGTH (L_s) = 7200'

$V_{ave} \simeq \frac{1.49}{.04} \times 3^{2/3} \times .005^{1/2} \simeq 5.5$ fps

T_{t_2} = 7200 / 5.5 = 1309 sec = 22 min = .36 hours

T_c = 1.3 + .36 = 1.66 hours

INDIAN FIELDS BRANCH

① AVERAGE LAND SLOPE ≃ 0.75 %   UPLAND LENGTH(L_u) = 52

VELOCITY ≃ .85 fps (NEARLY BARE GROUND)

T_{t_1} = 5200/.85 = 6118 sec ≃ 1.70 hours

② STREAM LENGTH (L_s) = 16000'

$V_{ave} = \frac{1.49}{.04} \times 3^{2/3} \times .0026^{1/2} = 3.95$ fps

T_{t_2} = 16000/3.95 = 4050 sec ≃ 1.13 hours

T_c = 1.70 + 1.13 = $\underline{\underline{2.83}}$ hours.

UNIT HYDROGRAPH COMPUTATION (USING BPR VALUES FOR T_c)

JACKSON RUN BRANCH

$T_p = \frac{D}{2} + .6T_c$ (D = .5 HR)

T_p = .25 + .6 × 1.5 = 1.15 HR.

T_b = 2.67 × T_p = 2.9 HR.

q_p = 484 A/T_p = 484 × 1.36/1.1 = 598 cfs

INDIAN FIELDS BRANCH

$T_p = \frac{D}{2} + .6 \times T_c$ (D = .5 HR)

T_p = .25 + .6 × 2.3 = 1.63 HR.

T_b = 2.67 × T_p = 4.35 HR.

q_p = 484 A/T_p = 484 × 5.9/1.63 = 1752

"PMP" RAINFALL INCREMENTS (MAX. 6 HRS.) D=.5HR.

TIME (HRS)	Σ% 6HR. PMF	Σ 6HR. PMP	INCREMENTAL PM
0.5	30	6.5"	6.5"
1.0	49	10.6	4.1
1.5	58	12.5	1.9
2.0	65	14.0	1.5
2.5	71	15.3	1.3
3.0	76	16.4	1.1
3.5	80	17.3	.9
4.0	84	18.1	.8
4.5	88	19.0	.9
5.0	92	19.9	.9
5.5	96	20.7	.8
6.0	100	21.6	.9

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ADJUSTED 6HR. SEQUENCE FOR A THIRD QUARTILE PEAK RUNOFF

TIME (HRS.)	INCREMENTAL PMP	Σ 6HR. PMP
.5	.8	.8
1.0	.8	1.6
1.5	.9	2.5
2.0	.9	3.4
2.5	1.1	4.5
3.0	1.3	5.8
3.5	4.1	9.9
4.0	6.5	16.4
4.5	1.9	18.3
5.0	1.5	19.8
5.5	.9	20.7
6.0	.9	21.6
	21.6 → ✓	

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SOIL COMPLEX NUMBER

BRIDGETON SOILS COMMONLY SANDS & GRAVELS, WITH SOME LOCAL SILTS AND CLAYS (SOIL GROUP B).

JACKSON RUN BRANCH

≃ 50%	COMMERCIAL & BUSINESS	CN 92
≃ 30%	RESIDENTIAL	CN 75
≃ 10%	MEADOWS	CN 58
≃ 10%	WOODS	CN 60

WEIGHTED CN:

$.5 \times 92 + .3 \times 75 + .1 \times 58 + .1 \times 60 = \underline{80}$

INDIAN FIELDS BRANCH

≃ 15%	COMMERCIAL & BUSINESS	CN 92
≃ 20%	RESIDENTIAL	CN 75
≃ 35%	MEADOWS	CN 58
≃ 25%	WOODS	CN 60
≃ 5%	SWAMP	CN 85

WEIGHTED CN:

$.15 \times 92 + .2 \times 75 + .35 \times 58 + .25 \times 60 + .05 \times 85 = \underline{68}$

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PROJECT East Lake Dam

JACKSON RUN BRANCH - DIRECT RUNOFF COMPUTATIONS

TIME (HR)	PMP RAINFALL INCR	PMP RAINFALL Σ	DIRECT RUNOFF INCR.	DIRECT RUNOFF Σ	LOSSES INCR.	LOSSES Σ
.5	.8	.8	0.0	0.0	.8	.8
1.0	.8	1.6	0.3	0.3	.5	1.3
1.5	.9	2.5	0.6	0.9	.3	1.6
2.0	.9	3.4	.7	1.6	.2	1.8
2.5	1.1	4.5	.9	2.5	.2	2.0
3.0	1.3	5.8	1.1	3.6	.2	2.2
3.5	4.1	9.9	3.8	7.4	.3	2.5
4.0	6.5	16.4	6.3	13.7	.2	2.7
4.5	1.9	18.3	1.8	15.5	.1	2.8 *
5.0	1.5	19.8	1.4	16.9	.1	2.9 *
5.5	.9	20.7	.8	17.7	.1	3.0 *
6.0	.9	21.6	.8	18.5 ✓	.1	3.1 *

* MINIMUM LOSS RATE .1" / HALF HOUR

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PROJECT EAST LAKE DAM

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INDIAN FIELDS BRANCH - DIRECT RUNOFF COMPUTATIONS

TIME (HR)	PMP RAINFALL INCR	PMP RAINFALL Σ	DIRECT RUNOFF INCR	DIRECT RUNOFF Σ	LOSSES INCR	LOSSES Σ
.5	.8	.8	0.0	0.0	0.8	0.8
1.0	.8	1.6	0.1	0.1	0.7	1.5
1.5	.9	2.5	0.3	0.4	0.6	2.1
2.0	.9	3.4	0.4	0.8	0.5	2.6
2.5	1.1	4.5	0.7	1.5	0.4	3.0
3.0	1.3	5.8	1.0	2.5	0.3	3.3
3.5	4.1	9.9	3.4	5.9	0.7	4.0
4.0	6.5	16.4	6.0	11.9	0.5	4.5
4.5	1.9	18.3	1.8	13.7	0.1	4.6*
5.0	1.5	19.8	1.4	15.1	0.1	4.7*
5.5	.9	20.7	.8	15.9	0.1	4.8*
6.0	.9	21.6	.8	16.7	0.1	4.9*

* MINIMUM LOSS RATE .1"/HALF HOUR

UNIT HYDROGRAPHS

JACKSON RUN BRANCH

T_p = 1.15 HOURS q_p = 598 cfs

T/Tp	q/qp	TIME (HRS)	DISCHARGE (CFS)	ADJUSTED q's FOR 1" RUNOFF
0.0	0.00	0.0	0	0
0.1	0.015	0.12	9	8
0.2	0.075	0.23	45	42
0.3	0.16	0.35	96	90
0.4	0.28	0.46	167	157
0.5	0.43	0.58	257	242
0.6	0.60	0.69	359	338
0.7	0.77	0.81	460	433
0.8	0.89	0.92	532	501
0.9	0.97	1.04	580	547
1.0	1.00	1.15	598	564
1.1	0.98	1.27	586	552
1.2	0.92	1.38	550	518
1.3	0.84	1.50	502	473
1.4	0.75	1.61	449	423
1.5	0.66	1.73	395	372
1.6	0.56	1.84	335	316
1.8	0.42	2.07	251	237
2.0	0.32	2.30	191	180
2.2	0.24	2.53	144	136
2.4	0.18	2.76	108	102
2.6	0.13	2.99	78	74
2.8	0.098	3.22	59	56
3.0	0.075	3.45	45	42
3.5	0.036	4.03	21	20
4.0	0.018	4.60	11	10
4.5	0.009	5.18	5	5
5.0	0.004	5.75	2	2
5.5	0.00	6.33	0	0

1"

$\sum_{t=0}^{1.84} q_t = 5752.5$ $\sum_{t=1.84}^{3.45} q_t = 1021$ $\sum_{t=3.45}^{6.33} q_t = 61.5$

RUNOFF (INCHES) = $\Sigma q \times \Delta t / (645.6 \times DA)$

= $(5752.5 \times .115 + 1021 \times .23 + 61.5 \times .575)/(645.6 \times 1.36)$

= $\underline{1.061''}$ adjust

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PROJECT East Lake Dam

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INDIAN FIELDS BRANCH

$T_P = 1.63$ HRS. $q_P = 1752$ cfs

T/Tp	q/qp	TIME (HOURS)	DISCHARGE (CFS)	ADJUSTED q's 1" RUNOFF
0.0	0.00	0.0	0	0
0.1	0.015	0.16	26	26
0.2	0.075	0.33	131	129
0.3	0.16	0.49	280	276
0.4	0.28	0.65	491	483
0.5	0.43	0.82	752	741
0.6	0.60	0.98	1051	1034
0.7	0.77	1.14	1349	1328
0.8	0.89	1.30	1559	1534
0.9	0.97	1.47	1699	1672
1.0	1.00	1.63	1752	1724
1.1	0.98	1.79	1717	1690
1.2	0.92	1.96	1612	1586
1.3	0.84	2.12	1472	1449
1.4	0.75	2.28	1314	1293
1.5	0.66	2.45	1156	1138
1.6	0.56	2.61	981	965
1.8	0.42	2.94	736	724
2.0	0.32	3.26	561	552
2.2	0.24	3.59	420	413
2.4	0.18	3.91	315	310
2.6	0.13	4.24	228	224
2.8	0.098	4.57	172	169
3.0	0.075	4.89	131	129
3.5	0.036	5.71	63	62
4.0	0.018	6.52	32	31
4.5	0.009	7.34	16	16
5.0	0.004	8.15	7	7
5.5	0.00	8.97	0	0

$\sum_{0}^{2.61} q = 16852.5$ $\sum_{2.61}^{4.89} q = 2988$ $\sum_{4.89}^{8.97} q = 183.5$

1"

RUNOFF (INCHES) = $\Sigma q \times \Delta t / 645.6 \times DA$

$= 16852.5 \times .163 + 2988 \times .326 + 183.5 \times .815 / 645.6 \times 5.9$

$= 1.016"$ adjust

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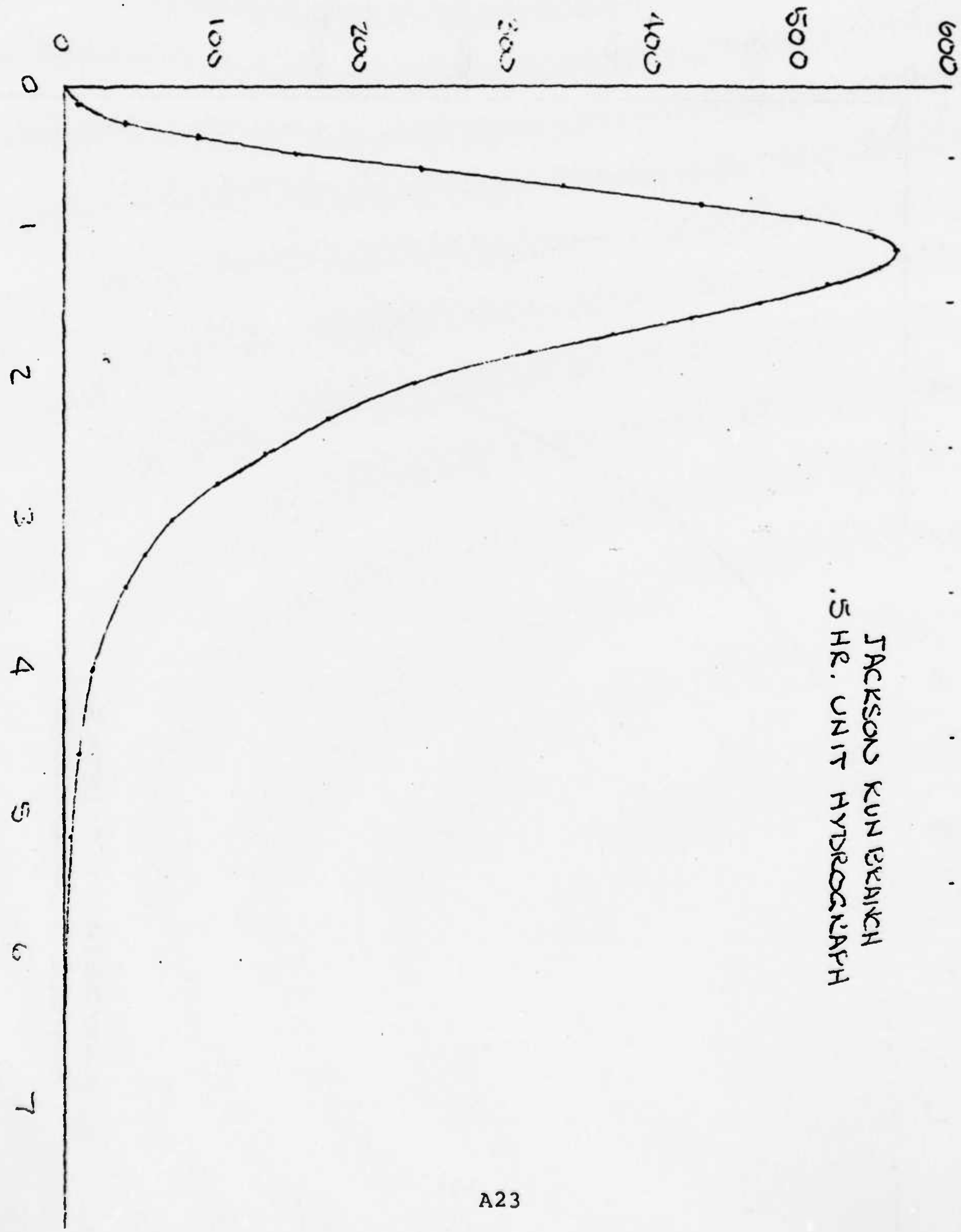

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PROJECT East Lake Dam

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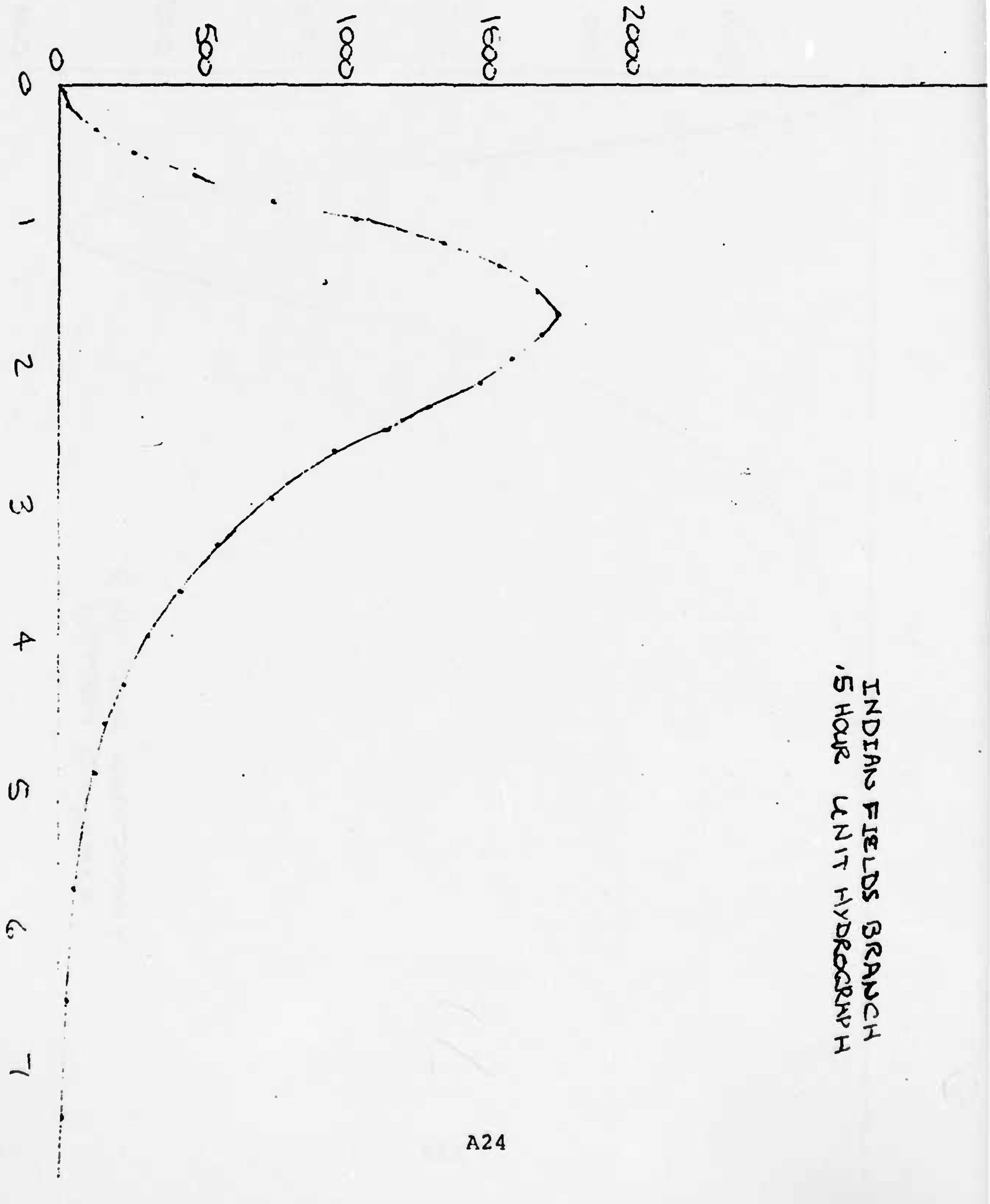

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NAME OF CLIENT CORPS OF ENGINEERS
PROJECT EAST LAKE DAM

— UNIT HYDROGRAPH —
JACKSON RUN BRANCH - ORDINATES FOR 5 MINUTE INCREMENTS

TIME (MIN)	q (CFS)	TIME (MIN)	q (CFS)	TIME (MIN)	q (CFS)
0	0	185	67	370	1
5	6	190	60	375	1
10	22	195	54	380	0
15	50	200	49		
20	83	205	44		
25	131	210	40		
30	185	215	37		
35	242	220	33		
40	318	225	30		
45	386	230	27		
50	447	235	23		
55	499	240	20		
60	532	245	19		
65	554	250	17		
70	562	255	16		
75	554	260	14		
80	532	265	13		
85	504	270	11		
90	473	275	10		
95	435	280	9		
100	399	285	8		
105	362	290	7		
110	317	295	7		
115	290	300	6		
120	261	305	5		
125	236	310	5		
130	213	315	4		
135	192	320	4		
140	177	325	3		
145	155	330	3		
150	139	335	3		
155	133	340	2		
160	120	345	2		
165	103	350	2		
170	94	355	2		
175	83	360	1		
180	73	365	1		

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SHEET NO. 14 OF ______

DATE 2/16/78

COMP. BY DEC

CHECKED BY LRC

NAME OF CLIENT Corps of Engineers

PROJECT East Lake Dam

— UNIT HYDROGRAPH —

INDIAN FIELDS BRANCH - ORDINATES FOR 5 MINUTE INCREMEN

TIME (MIN)	q (CFS)	TIME (MIN)	(q CFS)	TIME (MIN)	q (CFS)
0	0	185	649	370	44
5	13	190	606	375	41
10	28	195	563	380	38
15	81	200	526	385	35
20	129	205	489	390	32
25	206	210	452	395	30
30	285	215	415	400	29
35	397	220	389	405	27
40	500	225	363	410	26
45	635	230	337	415	24
50	765	235	311	420	22
55	918	240	289	425	21
60	1071	245	266	430	19
65	1224	250	244	435	18
70	1362	255	222	440	16
75	1470	260	208	445	15
80	1560	265	195	450	13
85	1629	270	181	455	12
90	1682	275	167	460	10
95	1709	280	157	465	9
100	1724	285	147	470	7
105	1695	290	137	475	5
110	1643	295	127	480	3
115	1613	300	120	485	1
120	1552	305	114	490	1
125	1480	210	107	495	0
130	1404	315	100		
135	1321	320	94		
140	1244	325	87		
145	1160	330	80		
150	1074	335	73		
155	994	340	67		
160	924	345	60		
165	862	350	57		
170	802	355	54		
175	741	360	51		
180	692	365	48		

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 15 OF

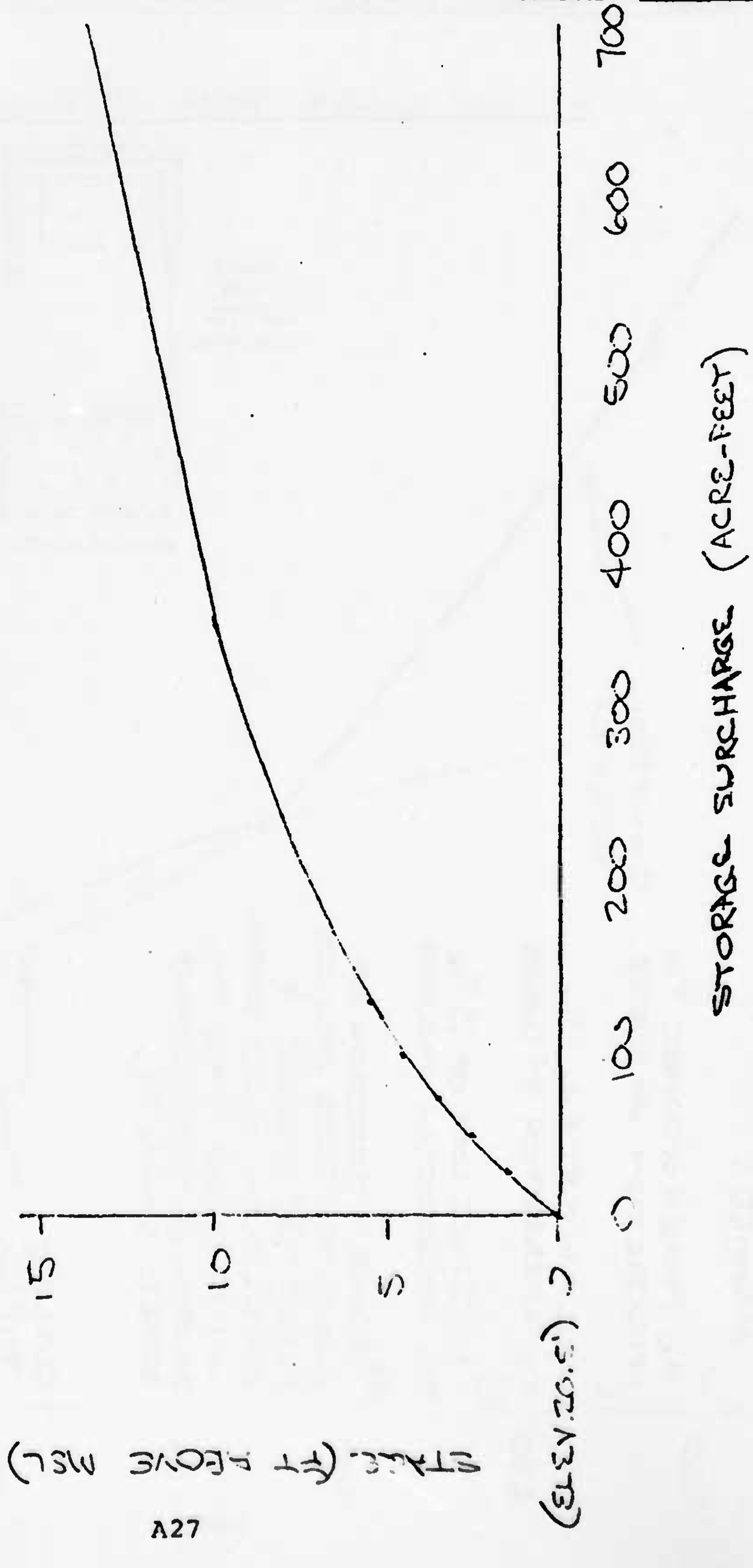
DATE 2/23/79

COMP. BY DFC

CHECKED BY LRW

NAME OF CLIENT CORPS OF ENGINEERS

PROJECT EAST LAKE DAM



JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 16 OF ____

DATE 2/28/78

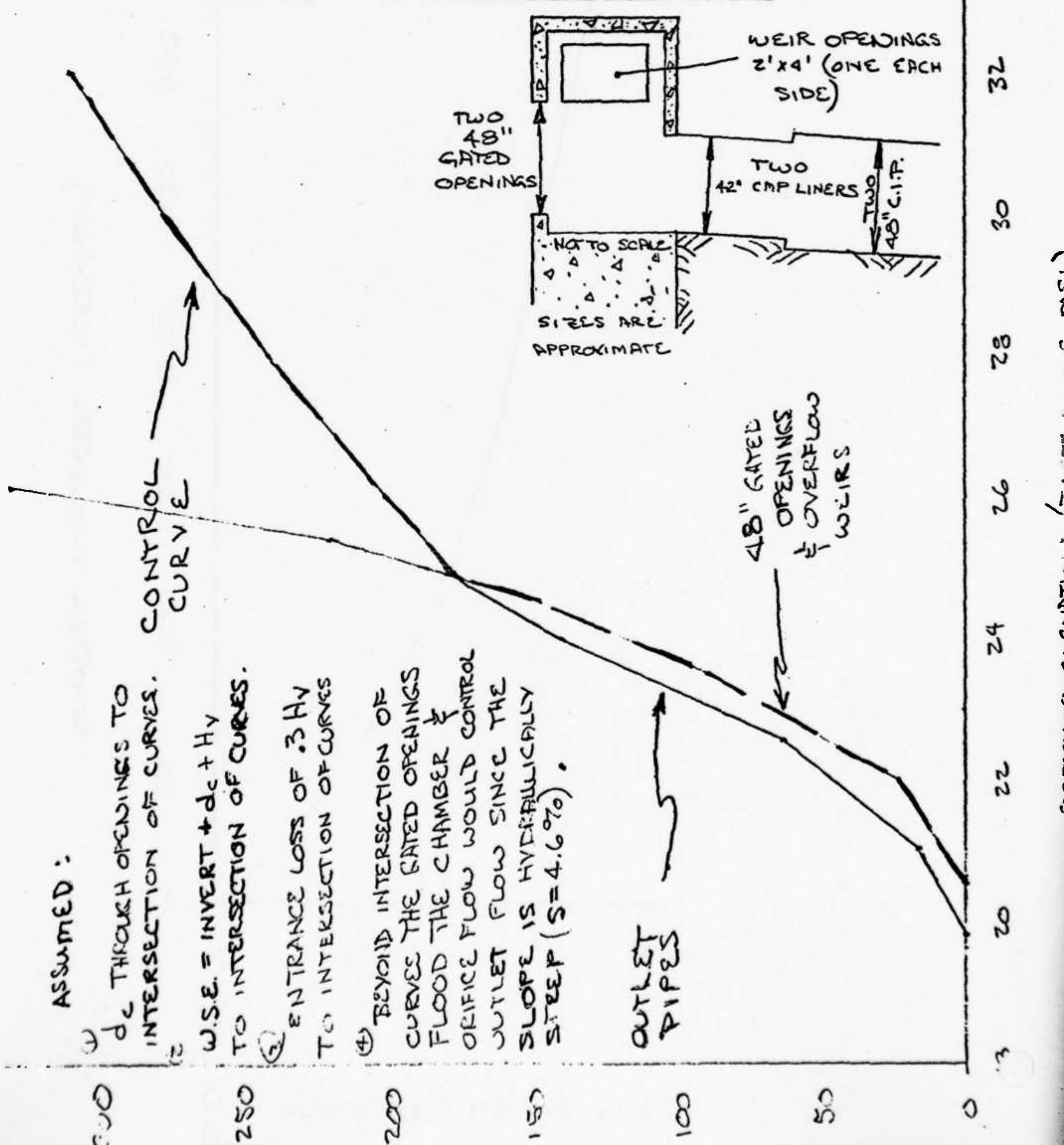
NAME OF CLIENT Corps of Engineers

COMP. BY DEC

PROJECT East Lake Dam

CHECKED BY LRW

RATING CURVE FOR EAST OUTLET



THE WEST OUTLET CHANNEL HAS A VERY MINOR CAPACITY AND CAN BE IGNORED IN CONJUNCTION WITH THE SPILLWAY DESIGN FLOOD. SHEET FLOW THROUGH THIS AREA IS CONSIDERED.

OVERFLOW

(2) STONE WALL ALONG COMMERCE STREET AND EXTENDING ACROSS POLE ESTATE.

L = 830'   ELEV ≈ 27.5' (MSL)   C = 3.0

(3) AREA TO NORTH SIDE OF POLE ESTATE INCLUDING THE WEST OUTLET (BEYOND THE WALL).

L ≈ 100'   ELEV ≈ 26.0 MSL   C ≈ 2.6

SEE PLOT ON NEXT PAGE

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 18 OF

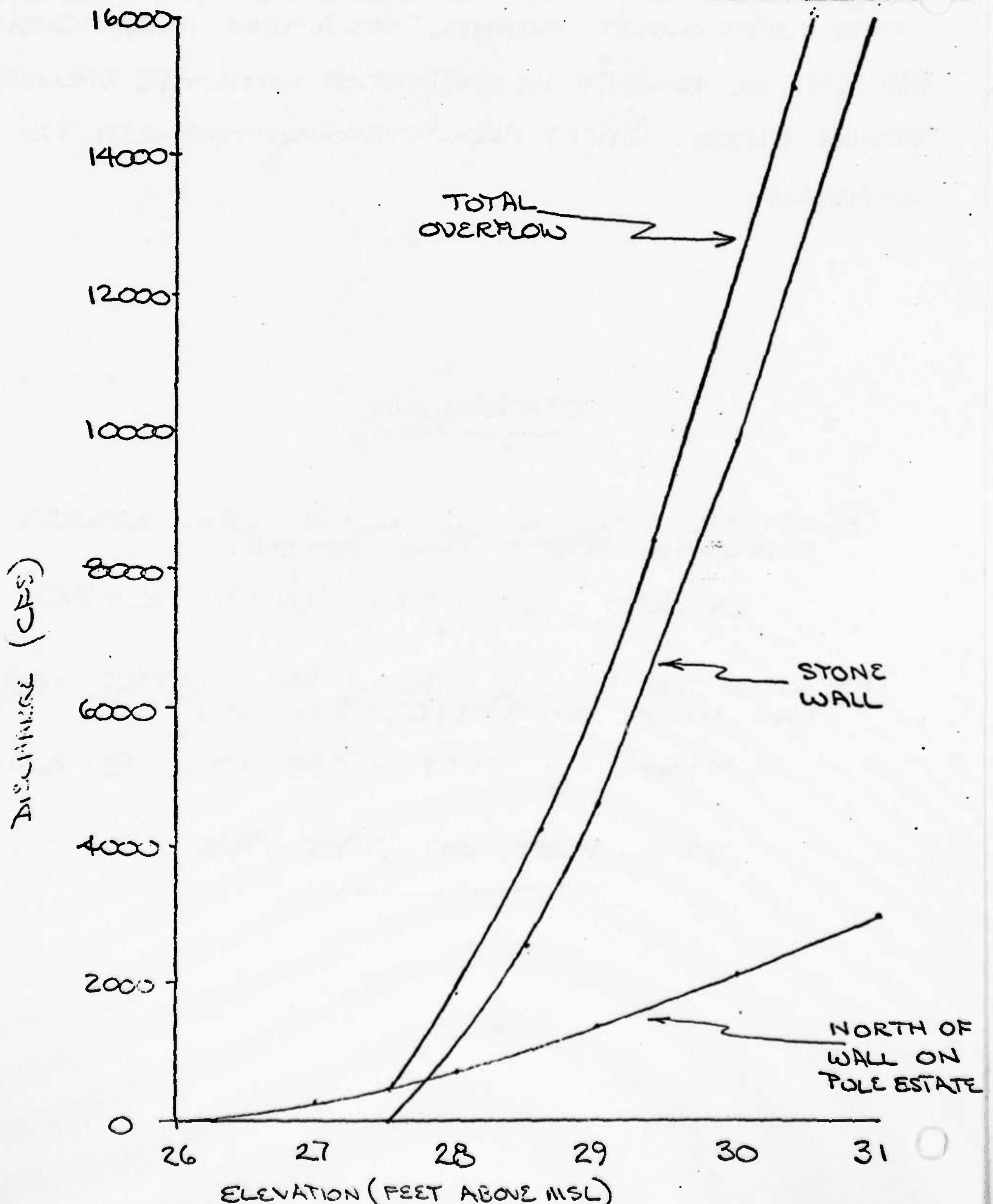
DATE 2/28/78

NAME OF CLIENT CORPS OF ENGINEERS

COMP. BY DBC

PROJECT EAST LAKE DAM

CHECKED BY LRW



Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 19 OF ______
DATE 2/28/78
COMP. BY DBC
CHECKED BY LRW

NAME OF CLIENT Corps of Engrs
PROJECT East Lake Dam

TOTAL STAGE - DISCHARGE RELATION

ELEV (MSL)	STAGE (FT)	EAST OUTLET	TOTAL OVERFLOW	TOTAL
20.5	0	0	0	0
22.0	1.5	20	0	20
24.0	3.5	120	0	120
26.0	5.5	200	0	200
27.0	6.5	220	260	480
28.0	7.5	240	2000	2240
29.0	8.5	260	6000	6260
29.5	9.0	270	9000	9270
30.0	9.5	280	12400	12680
30.5	10.0	285	16000	16285

SHEET NO. 20 OF

DATE 2/28/78

NAME OF CLIENT CORPS OF ENGRS.

COMP. BY DBC

PROJECT EAST LAKE DAM

CHECKED BY LRW

STORAGE-DISCHARGE RELATION

SURCHARGE STAGE	SURCHARGE STORAGE	DISCHARGE
0	0	0
1.5	25	20
3.5	68	120
5.5	125	200
6.5	170	480
7.5	210	2240
8.5	260	6260
9.0	290	9270
9.5	320	12680
10.0	350	16285

HEC-1 COMPUTATIONS

```
****************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****************************

                         NATIONAL DAM SAFETY PROGRAM
                                EAST LAKE DAM
                   ONE HALF PROBABLE MAXIMUM FLOOD COMPUTATION

                             JOB SPECIFICATION
        NQ    NHR   NMIN   IDAY    IHR   IMIN  METRC   IPLT   IPRT  NSTAN
       150      0      5      1      0      0      0      2      2      0
                             JOPER    NWT
                                 3      0

   **********    **********    **********    **********    **********

                         SUB-AREA RUNOFF COMPUTATION

        HYDROGRAPH COMPUTATION FOR THE JACKSON RUN BRANCH,
              ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPPT   INAME
                  1       0       0       0       0       0       1

                              HYDROGRAPH DATA
      IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
          0      -1    1.36    0.00    0.00    0.00   0.000       0       0       0

                                PRECIP DATA
                          NP   STORM     DAJ     DAK
                          61    0.00    0.00    0.00
                              PRECIP PATTERN
     .15    0.00    0.00    0.00    0.00    0.00     .30    0.00    0.00    0.00
    0.00    0.00     .35    0.00    0.00    0.00    0.00    0.00     .45    0.00
    0.00    0.00    0.00    0.00     .55    0.00    0.00    0.00    0.00    0.00
    1.90    0.00    0.00    0.00    0.07    0.00    3.15    0.00    0.00    0.00
    0.00    0.00     .90    0.00    0.00    0.00    0.00    0.00     .70    0.00
    0.00    0.00    0.00    0.00     .40    0.00    0.00    0.00    0.00    0.00
     .40

                                 LOSS DATA
       STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
        0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                        GIVEN UNIT GRAPH, NUHGQ=   77
      0.      6.     22.     50.     93.    131.    185.    242.    318.    386.
    447.    493.    532.    554.    563.    554.    532.    504.    473.    435.
    399.    362.    317.    290.    261.    236.    213.    192.    177.    158.
    139.    133.    120.    103.     94.     83.     73.     67.     60.     54.
     49.     44.     40.     37.     33.     30.     27.     23.     20.     19.
     17.     16.     14.     13.     11.     10.      9.      9.      7.      7.
      6.      5.      5.      4.      4.      3.      3.      3.      2.      2.
      2.      2.      1.      1.      1.      1.      0.
             UNIT GRAPH TOTALS  10526. CFS OR 1.00 INCHES OVER THE AREA

                              RECESSION DATA
                 STRTQ=    0.00    QRCSN=    0.00    RTIOR= 1.00
```

1	0	15	0.00	0.00	3.
1	0	20	0.00	0.00	5.
1	0	25	0.00	0.00	12.
1	0	30	0.00	0.00	20.
1	0	35	.30	.30	28.
1	0	40	0.00	0.00	38.
1	0	45	0.00	0.00	54.
1	0	50	0.00	0.00	73.
1	0	55	0.00	0.00	92.
1	0	60	0.00	0.00	114.
1	1	5	.35	.35	135.
1	1	10	0.00	0.00	158.
1	1	15	0.00	0.00	188.
1	1	20	0.00	0.00	216.
1	1	25	0.00	0.00	243.
1	1	30	0.00	0.00	271.
1	1	35	.45	.45	295.
1	1	40	0.00	0.00	319.
1	1	45	0.00	0.00	350.
1	1	50	0.00	0.00	378.
1	1	55	0.00	0.00	401.
1	1	60	0.00	0.00	428.
1	2	5	.55	.55	451.
1	2	10	0.00	0.00	472.
1	2	15	0.00	0.00	504.
1	2	20	0.00	0.00	533.
1	2	25	0.00	0.00	555.
1	2	30	0.00	0.00	584.
1	2	35	1.90	1.90	606.
1	2	40	0.00	0.00	637.
1	2	45	0.00	0.00	692.
1	2	50	0.00	0.00	756.
1	2	55	0.00	0.00	821.
1	2	60	0.00	0.00	911.
1	3	5	3.15	3.15	1001.
1	3	10	0.00	0.00	1112.
1	3	15	0.00	0.00	1282.
1	3	20	0.00	0.00	1465.
1	3	25	0.00	0.00	1644.
1	3	30	0.00	0.00	1855.
1	3	35	.90	.90	2048.
1	3	40	0.00	0.00	2238.
1	3	45	0.00	0.00	2471.
1	3	50	0.00	0.00	2656.
1	3	55	0.00	0.00	2799.
1	3	60	0.00	0.00	2924.
1	4	5	.70	.70	2988.
1	4	10	0.00	0.00	3021.
1	4	15	0.00	0.00	3038.
1	4	20	0.00	0.00	2998.
1	4	25	0.00	0.00	2906.
1	4	30	0.00	0.00	2828.
1	4	35	.40	.40	2725.
1	4	40	0.00	0.00	2613.
1	4	45	0.00	0.00	2511.
1	4	50	0.00	0.00	2391.
1	4	55	0.00	0.00	2248.
1	4	60	0.00	0.00	2147.
1	5	5	.40	.40	2028.
1	5	10	0.00	0.00	1939.
1	5	15	0.00	0.00	1845.
1	5	20	0.00	0.00	1740.
1	5	25	0.00	0.00	1653.
[illegible]	5	30	[illegible]	[illegible]	[illegible]

1	5	35	0.00	0.00	1466.
1	5	40	0.00	0.00	1416.
1	5	45	0.00	0.00	1346.
1	5	50	0.00	0.00	1257.
1	5	55	0.00	0.00	1187.
1	5	60	0.00	0.00	1114.
1	6	5	0.00	0.00	1035.
1	6	10	0.00	0.00	980.
1	6	15	0.00	0.00	909.
1	6	20	0.00	0.00	835.
1	6	25	0.00	0.00	767.
1	6	30	0.00	0.00	697.
1	6	35	0.00	0.00	631.
1	6	40	0.00	0.00	584.
1	6	45	0.00	0.00	527.
1	6	50	0.00	0.00	474.
1	6	55	0.00	0.00	426.
1	6	60	0.00	0.00	380.
1	7	5	0.00	0.00	336.
1	7	10	0.00	0.00	311.
1	7	15	0.00	0.00	280.
1	7	20	0.00	0.00	252.
1	7	25	0.00	0.00	228.
1	7	30	0.00	0.00	205.
1	7	35	0.00	0.00	179.
1	7	40	0.00	0.00	166.
1	7	45	0.00	0.00	150.
1	7	50	0.00	0.00	133.
1	7	55	0.00	0.00	119.
1	7	60	0.00	0.00	107.
1	8	5	0.00	0.00	94.
1	8	10	0.00	0.00	86.
1	8	15	0.00	0.00	77.
1	8	20	0.00	0.00	69.
1	8	25	0.00	0.00	63.
1	8	30	0.00	0.00	55.
1	8	35	0.00	0.00	48.
1	8	40	0.00	0.00	45.
1	8	45	0.00	0.00	39.
1	8	50	0.00	0.00	36.
1	8	55	0.00	0.00	31.
1	8	60	0.00	0.00	28.
1	9	5	0.00	0.00	22.
1	9	10	0.00	0.00	21.
1	9	15	0.00	0.00	19.
1	9	20	0.00	0.00	17.
1	9	25	0.00	0.00	13.
1	9	30	0.00	0.00	12.
1	9	35	0.00	0.00	10.
1	9	40	0.00	0.00	9.
1	9	45	0.00	0.00	8.
1	9	50	0.00	0.00	7.
1	9	55	0.00	0.00	6.
1	9	60	0.00	0.00	5.
1	10	5	0.00	0.00	4.
1	10	10	0.00	0.00	4.
1	10	15	0.00	0.00	4.
1	10	20	0.00	0.00	3.
1	10	25	0.00	0.00	2.
1	10	30	0.00	0.00	2.
1	10	35	0.00	0.00	2.
1	10	40	0.00	0.00	2.
1	10	45	0.00	0.00	1.
1	10	50	0.00	0.00	1.

```
*DVV*

**********     **********     **********     **********     **********

                         SUB-AREA RUNOFF COMPUTATION

HYDROGRAPH COMPUTATION FOR THE INDIAN FIELDS BRANCH.
                  ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                      2       0       0       0       0       0       1

                                HYDROGRAPH DATA
        IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
            0      -1    5.90    0.00    0.00    0.00   0.000       0       0       0

                                  PRECIP DATA
                              NP   STORM     DAJ     DAK
                              61    0.00    0.00    0.00
                                 PRECIP PATTERN
    .05    0.00    0.00    0.00    0.00    0.00     .15    0.00    0.00    0.00
   0.00    0.00     .20    0.00    0.00    0.00    0.00    0.00     .35    0.00
   0.00    0.00    0.00    0.00     .50    0.00    0.00    0.00    0.00    0.00
   1.70    0.00    0.00    0.00    0.00    0.00    3.00    0.00    0.00    0.00
   0.00    0.00     .90    0.00    0.00    0.00    0.00    0.00     .70    0.00
   0.00    0.00    0.00    0.00     .40    0.00    0.00    0.00    0.00    0.00
    .40

                                   LOSS DATA
        STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
         0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                         GIVEN UNIT GRAPH, NUHGQ= 100
     0.     13.     28.     81.    129.    206.    285.    397.    500.    635.
   765.    918.   1071.   1224.   1362.   1470.   1560.   1629.   1682.   1709.
  1724.   1695.   1663.   1613.   1552.   1480.   1404.   1321.   1244.   1168.
  1084.    994.    924.    863.    802.    741.    692.    649.    606.    563.
   526.    489.    452.    415.    389.    363.    337.    311.    289.    266.
   244.    222.    208.    195.    181.    167.    157.    147.    137.    127.
   120.    114.    107.    100.     94.     87.     80.     73.     67.     60.
    57.     54.     51.     48.     44.     41.     38.     35.     32.     30.
    23.     27.     26.     24.     22.     21.     19.     18.     16.     15.
    13.     12.     10.      9.      7.      5.      3.      1.      1.      0.
              UNIT GRAPH TOTALS  45678. CFS OR 1.00 INCHES OVER THE AREA

                                 RECESSION DATA
                  STRTQ=    0.00   QRCSN=    0.00   RTIOR= 1.00

                              END-OF-PERIOD FLOW
                            TIME    RAIN    EXCS    COMP Q
                          1  0  5     .05     .05       0.
                          1  0 10    0.00    0.00       1.
                          1  0 15    0.00    0.00       1.
                          1  0 20    0.00    0.00       4.
                          1  0 25    0.00    0.00       6.
                          1  0 30    0.00    0.00      10.
                          1  0 35     .15     .15      14.
                          1  0 40    0.00    0.00      22.
                          1  0 45    0.00    0.00      33.
                          1  0 50    0.00    0.00      44.
                          1  0 55    0.00    0.00      58.
```

A36

1	1	5	.20	.20	96.
1	1	10	0.00	0.00	123.
1	1	15	0.00	0.00	149.
1	1	20	0.00	0.00	185.
1	1	25	0.00	0.00	219.
1	1	30	0.00	0.00	260.
1	1	35	.35	.35	302.
1	1	40	0.00	0.00	353.
1	1	45	0.00	0.00	400.
1	1	50	0.00	0.00	461.
1	1	55	0.00	0.00	515.
1	1	60	0.00	0.00	581.
1	2	5	.50	.50	644.
1	2	10	0.00	0.00	721.
1	2	15	0.00	0.00	790.
1	2	20	0.00	0.00	877.
1	2	25	0.00	0.00	956.
1	2	30	0.00	0.00	1050.
1	2	35	1.70	1.70	1141.
1	2	40	0.00	0.00	1263.
1	2	45	0.00	0.00	1376.
1	2	50	0.00	0.00	1550.
1	2	55	0.00	0.00	1707.
1	2	60	0.00	0.00	1914.
1	3	5	3.00	3.00	2116.
1	3	10	0.00	0.00	2402.
1	3	15	0.00	0.00	2668.
1	3	20	0.00	0.00	3073.
1	3	25	0.00	0.00	3445.
1	3	30	0.00	0.00	3927.
1	3	35	.90	.90	4403.
1	3	40	0.00	0.00	4973.
1	3	45	0.00	0.00	5489.
1	3	50	0.00	0.00	6062.
1	3	55	0.00	0.00	6586.
1	3	60	0.00	0.00	7161.
1	4	5	.70	.70	7706.
1	4	10	0.00	0.00	8237.
1	4	15	0.00	0.00	8704.
1	4	20	0.00	0.00	9060.
1	4	25	0.00	0.00	9355.
1	4	30	0.00	0.00	9597.
1	4	35	.40	.40	9775.
1	4	40	0.00	0.00	9983.
1	4	45	0.00	0.00	9942.
1	4	50	0.00	0.00	9872.
1	4	55	0.00	0.00	9785.
1	4	60	0.00	0.00	9655.
1	5	5	.40	.40	9471.
1	5	10	0.00	0.00	9242.
1	5	15	0.00	0.00	9016.
1	5	20	0.00	0.00	8751.
1	5	25	0.00	0.00	8489.
1	5	30	0.00	0.00	8221.
1	5	35	0.00	0.00	7930.
1	5	40	0.00	0.00	7617.
1	5	45	0.00	0.00	7348.
1	5	50	0.00	0.00	7070.
1	5	55	0.00	0.00	6800.

1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	6	45	0.00	0.00	4163.
1	6	50	0.00	0.00	3912.
1	6	55	0.00	0.00	3682.
1	6	60	0.00	0.00	3447.
1	7	5	0.00	0.00	3224.
1	7	10	0.00	0.00	2997.
1	7	15	0.00	0.00	2795.
1	7	20	0.00	0.00	2594.
1	7	25	0.00	0.00	2426.
1	7	30	0.00	0.00	2260.
1	7	35	0.00	0.00	2102.
1	7	40	0.00	0.00	1945.
1	7	45	0.00	0.00	1815.
1	7	50	0.00	0.00	1688.
1	7	55	0.00	0.00	1575.
1	7	60	0.00	0.00	1461.
1	8	5	0.00	0.00	1362.
1	8	10	0.00	0.00	1269.
1	8	15	0.00	0.00	1183.
1	8	20	0.00	0.00	1095.
1	8	25	0.00	0.00	1025.
1	8	30	0.00	0.00	953.
1	8	35	0.00	0.00	885.
1	8	40	0.00	0.00	817.
1	8	45	0.00	0.00	759.
1	8	50	0.00	0.00	698.
1	8	55	0.00	0.00	654.
1	8	60	0.00	0.00	609.
1	9	5	0.00	0.00	568.
1	9	10	0.00	0.00	529.
1	9	15	0.00	0.00	490.
1	9	20	0.00	0.00	453.
1	9	25	0.00	0.00	424.
1	9	30	0.00	0.00	393.
1	9	35	0.00	0.00	363.
1	9	40	0.00	0.00	339.
1	9	45	0.00	0.00	317.
1	9	50	0.00	0.00	294.
1	9	55	0.00	0.00	275.
1	9	60	0.00	0.00	255.
1	10	5	0.00	0.00	234.
1	10	10	0.00	0.00	219.
1	10	15	0.00	0.00	201.
1	10	20	0.00	0.00	186.
1	10	25	0.00	0.00	170.
1	10	30	0.00	0.00	156.
1	10	35	0.00	0.00	139.
1	10	40	0.00	0.00	126.
1	10	45	0.00	0.00	114.
1	10	50	0.00	0.00	103.
1	10	55	0.00	0.00	92.
1	10	60	0.00	0.00	81.
1	11	5	0.00	0.00	69.
1	11	10	0.00	0.00	60.
1	11	15	0.00	0.00	55.
1	11	20	0.00	0.00	48.
1	11	25	0.00	0.00	43.
1	11	30	0.00	0.00	39.
1	11	35	0.00	0.00	33.
1	11	40	0.00	0.00	30.
1	11	45	0.00	0.00	27.
1	11	50	0.00	0.00	24.
1	11	55	0.00	0.00	22.

"OVN"

*********** *********** *********** *********** ***********

COMBINE HYDROGRAPHS

COMBINED INFLOW HYDROGRAPH INTO EAST LAKE RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
3	2	0	0	0	0	1

SUM OF 2 HYDROGRAPHS AT 3

0.	2.	5.	12.	19.	30.	42.	60.	83.	117.
150.	191.	232.	281.	336.	401.	462.	532.	597.	672.
750.	839.	916.	1009.	1094.	1193.	1294.	1410.	1511.	1634.
1747.	1893.	2068.	2306.	2528.	2826.	3117.	3513.	3950.	4537.
5088.	5782.	6451.	7211.	7961.	8718.	9385.	10085.	10593.	11258.
11741.	12058.	12260.	12425.	12500.	12496.	12453.	12264.	12034.	11803.
11499.	11181.	10861.	10491.	10142.	9784.	9397.	9033.	8694.	8327.
7987.	7632.	7289.	6963.	6648.	6302.	5981.	5643.	5311.	4983.
4690.	4386.	4108.	3827.	3560.	3308.	3075.	2846.	2654.	2465.
2281.	2111.	1365.	1821.	1694.	1568.	1456.	1355.	1260.	1164.
1088.	1003.	933.	863.	797.	734.	685.	638.	590.	550.
509.	470.	437.	405.	373.	348.	325.	301.	281.	260.
238.	223.	205.	189.	173.	158.	140.	128.	115.	104.
92.	82.	70.	60.	55.	48.	43.	39.	33.	30.
27.	24.	22.	19.	16.	14.	12.	11.	9.	8.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12500.	6282.	3192.	3192.	478750.
INCHES		8.05	8.52	8.52	8.52
AC-FT		3117.	3299.	3299.	3299.

'OVN'

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ROUTED HYDROGRAPH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
4	1	0	0	0	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.000	0.00	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
0	0	0	0.000	0.000	0.000	-1.

STORAGE=	0.	25.	65.	125.	170.	210.	260.	290.	320.	350.
OUTFLOW=	0.	20.	120.	200.	480.	2240.	6260.	9270.	12680.	15285.

	TIME		EOP STOR	AVG IN	EOP OUT
1	0	5	0.	0.	0.
1	0	10	0.	1.	0.
1	0	15	0.	3.	0.
1	0	20	0.	8.	0.
1	0	25	0.	15.	0.
1	0	30	0.	24.	0.
1	0	35	1.	36.	0.
1	0	40	1.	51.	1.
1	0	45	1.	72.	1.
1	0	50	2.	100.	2.
1	0	55	3.	133.	2.
1	0	60	4.	170.	3.
1	1	5	6.	211.	4.
1	1	10	7.	256.	6.
1	1	15	9.	309.	8.
1	1	20	12.	369.	10.
1	1	25	15.	431.	12.
1	1	30	18.	497.	14.
1	1	35	22.	564.	18.
1	1	40	26.	634.	23.
1	1	45	31.	711.	34.
1	1	50	36.	794.	46.
1	1	55	42.	877.	59.
1	1	60	48.	963.	73.
1	2	5	55.	1052.	89.
1	2	10	62.	1143.	106.
1	2	15	70.	1243.	122.
1	2	20	78.	1352.	134.
1	2	25	87.	1460.	147.
1	2	30	97.	1572.	160.
1	2	35	107.	1690.	175.
1	2	40	119.	1923.	191.
1	2	45	131.	1983.	236.
1	2	50	144.	2187.	318.
1	2	55	159.	2417.	406.
1	2	60	173.	2677.	614.
1	3	5	187.	2971.	1234.
1	3	10	200.	3315.	1782.
1	3	15	211.	3732.	2331.

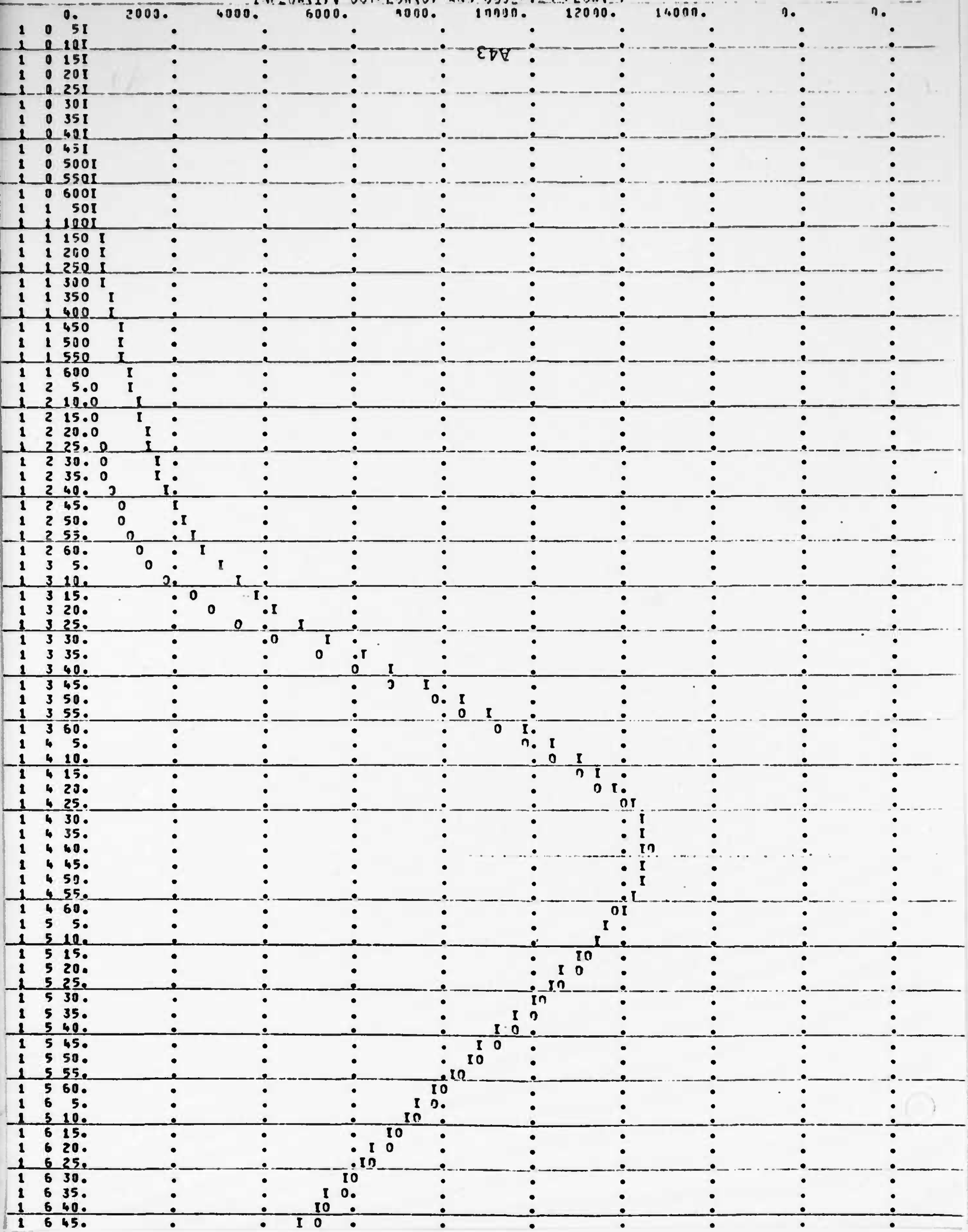
A40

1	3 25	230.	4913.	3377.
1	3 30	239.	5435.	4553.
1	3 35	247.	6116.	5231.
1	3 40	256.	6831.	5925.
1	3 45	265.	7586.	6716.
1	3 50	273.	8339.	7550.
1	3 55	281.	9052.	8321.
1	3 60	288.	9735.	9047.
1	4 5	294.	10389.	9781.
1	4 10	300.	10975.	10453.
1	4 15	306.	11499.	11042.
1	4 20	310.	11900.	11524.
1	4 25	313.	12159.	11881.
1	4 30	315.	12343.	12141.
1	4 35	317.	12462.	12322.
1	4 40	318.	12498.	12421.
1	4 45	318.	12474.	12451.
1	4 50	318.	12358.	12399.
1	4 55	316.	12149.	12258.
1	4 60	315.	11918.	12067.
1	5 5	313.	11651.	11833.
1	5 10	310.	11340.	11556.
1	5 15	307.	11021.	11255.
1	5 20	305.	10676.	10929.
1	5 25	302.	10317.	10585.
1	5 30	298.	9963.	10235.
1	5 35	295.	9590.	9872.
1	5 40	292.	9215.	9502.
1	5 45	289.	8863.	9154.
1	5 50	286.	8511.	8824.
1	5 55	282.	8157.	8481.
1	5 60	279.	7809.	8136.
1	6 5	275.	7460.	7789.
1	6 10	272.	7126.	7449.
1	6 15	269.	6806.	7119.
1	6 20	265.	6475.	6788.
1	6 25	262.	6141.	6456.
1	6 30	259.	5812.	6146.
1	6 35	255.	5477.	5856.
1	6 40	251.	5147.	5549.
1	6 45	247.	4836.	5240.
1	6 50	244.	4538.	4935.
1	6 55	240.	4247.	4637.
1	6 60	236.	3967.	4346.
1	7 5	233.	3694.	4063.
1	7 10	229.	3439.	3791.
1	7 15	226.	3192.	3531.
1	7 20	223.	2961.	3284.
1	7 25	220.	2750.	3052.
1	7 30	217.	2559.	2838.
1	7 35	215.	2373.	2637.
1	7 40	213.	2196.	2446.
1	7 45	210.	2038.	2269.
1	7 50	208.	1893.	2158.
1	7 55	206.	1759.	2053.
1	7 60	203.	1631.	1942.
1	8 5	201.	1512.	1829.
1	8 10	198.	1405.	1717.
1	8 15	196.	1307.	1609.
1	8 20	193.	1212.	1505.
1	8 25	191.	1120.	1405.
1	8 30	183.	1049.	1311.

1	9	5	177.	614.	783.	
1	9	10	176.	570.	727.	
1	9	15	174.	529.	675.	
1	9	20	173.	487.	626.	
1	9	25	172.	453.	581.	
1	9	30	171.	421.	539.	
1	9	35	170.	389.	499.	
1	9	40	170.	360.	477.	
1	9	45	169.	336.	471.	
1	9	50	168.	313.	465.	
1	9	55	166.	291.	457.	
1	9	60	165.	271.	450.	
1	10	5	164.	249.	441.	
1	10	10	162.	231.	432.	
1	10	15	161.	214.	423.	
1	10	20	159.	197.	414.	
1	10	25	158.	181.	406.	
1	10	30	156.	165.	394.	
1	10	35	155.	149.	384.	
1	10	40	153.	134.	373.	
1	10	45	151.	121.	363.	
1	10	50	149.	109.	352.	
1	10	55	148.	98.	341.	
1	10	60	146.	87.	331.	
1	11	5	144.	76.	320.	
1	11	10	143.	65.	309.	
1	11	15	141.	58.	299.	
1	11	20	139.	52.	288.	
1	11	25	138.	46.	278.	
1	11	30	136.	41.	268.	
1	11	35	134.	36.	258.	
1	11	40	133.	32.	249.	
1	11	45	131.	28.	240.	
1	11	50	130.	26.	231.	
1	11	55	129.	23.	222.	
1	11	60	127.	20.	214.	
1	12	5	126.	18.	205.	
1	12	10	125.	15.	199.	
1	12	15	123.	13.	198.	
1	12	20	122.	12.	196.	
1	12	25	121.	10.	194.	
1	12	30	119.	9.	192.	
		SUM			461496.	

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	12451.	6149.	3077.	3077.	461496.
INCHES		7.88	8.21	8.21	8.21
AC-FT		3051.	3180.	3181.	3180.

INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)



RECOMMENDED GUIDELINES

FOR SAFETY INSPECTION OF DAMS

CHAPTER 4

CHAPTER 4 - PHASE II INVESTIGATION

4.1. Purpose. The Phase II investigation will be supplementary to Phase I and should be conducted when the results of the Phase I investigation indicate the need for additional in-depth studies, investigations or analyses.

4.2. Scope. The Phase II investigation should include all additional studies, investigations and analyses necessary to evaluate the safety of the dam. Included, as required, will be additional visual inspections, measurements, foundation exploration and testing, materials testing, hydraulic and hydrologic analysis and structural stability analyses.

4.3. Hydraulic and Hydrologic Analysis. Hydraulic and hydrologic capabilities should be determined using the following criteria and procedures. Depending on the project characteristics, either the spillway design flood peak inflow or the spillway design flood hydrograph should be the basis for determining the maximum water surface elevation and maximum outflow. If the operation or failure of upstream water control projects would have significant impact on peak flow or hydrograph analyses, the impact should be assessed.

4.3.1. Maximum Water Surface Based on SDF Peak Inflow. When the total project discharge capability at maximum pool exceeds the peak inflow of the recommended SDF, and operational constraints would not prevent such a release at controlled projects, a reservoir routing is not required. The maximum discharge should be assumed equal to the peak inflow of the spillway design flood. Flood volume is not controlling in this situation and surcharge storage is either absent or is significant only to the extent that it provides the head necessary to develop the release capability required.

4.3.1.1. Peak for 100-Year Flood. When the 100-year flood is applicable under the provisions of Table 3 and data are available, the spillway design flood peak inflow may be determined by use of "A Uniform Technique for Determining Flood Frequencies," Water Resources Council (WRC), Hydrology Committee, Bulletin 15, December 1967. Flow frequency information from regional analysis is generally preferred over single station results when available and appropriate. Rainfall-runoff techniques may be necessary when there are inadequate runoff data available to make a reasonable estimate of flow frequency.

4.3.1.2. Peak for PMF or Fraction Thereof. When either the Probable Maximum Flood peak or a fraction thereof is applicable under the provisions of Table 3, the unit hydrograph - infiltration loss technique is generally the most expeditious method of computing the spillway design flood peak for most projects. This technique is discussed in the following paragraph.

4.3.2. Maximum Water Surface Based on SDF Hydrograph. Both peak and volume are required in this analysis. Where surcharge storage is significant, or where there is insufficient discharge capability at maximum pool to pass the peak inflow of the SDF, considering all possible operational constraints, a flood hydrograph is required. When there are upstream hazard areas that would be imperiled by fast rising reservoirs levels, SDF hydrographs should be routed to ascertain available time for warning and escape. Determination of probable maximum precipitation or 100-year precipitation, whichever is applicable, and unit hydrographs or runoff models will be required, followed by the determination of the PMF or 100-year flood. Conservative loss rates (significantly reduced by antecedent rainfall conditions where appropriate) should be estimated for computing the rainfall excess to be utilized with unit hydrographs. Rainfall values are usually arranged with gradually ascending and descending rates with the maximum rate late in the storm. When applicable, conservatively high snowmelt runoff rates and appropriate releases from upstream projects should be assumed. The PMP may be obtained from National Weather Service (NWS) publications such as Hydrometeorological Report (HMR) 33. Special NWS publications for particular areas should be used when available. Rainfall for the 100-year frequency flood can be obtained from the NWS publication "Rainfall Frequency Atlas of the United States," Technical Paper No. 40; Atlas 2, "Precipitation Frequency Atlas of Western United States;" or other NWS publications. The maximum water surface elevation and spillway design flood outflow are then determined by routing the inflow hydrograph through the reservoir surcharge storage, assuming a starting water surface at the bottom of surcharge storage, or lower when appropriate. For projects where the bottom of surcharge space is not distinct, or the flood control storage space (exclusive of surcharge) is appreciable, it may be appropriate to select starting water surface elevations below the top of the flood control storage for routings. Conservatively high starting levels should be estimated on the basis of hydrometeorological conditions reasonably characteristic for the region and flood release capability of the project. Necessary adjustment of reservoir storage capacity due to existing or future sediment or other encroachment may be approximated when accurate determination of deposition is not practicable.

4.3.3. Acceptable Procedures. Techniques for performing hydraulic and hydrologic analyses are generally available from publications prepared by Federal agencies involved in water resources development or textbooks written by the academic community. Some of these procedures are rather sophisticated and require expensive computational equipment and large data banks. While results of such procedures are generally more reliable than simplified methods, their use is generally not warranted in studies connected with this program unless they can be performed quickly and inexpensively. There may be situations where the more complex techniques have to be employed to obtain reliable results; however, these cases will be exceptions rather than the rule. Whenever the acceptability of procedures is in question, the advice of competent experts should be sought. Such expertise is generally available in the Corps of Engineers, Bureau of

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic

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information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. Clay Shale Foundation. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear stength. Prediction of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

TABLE 4

FACTORS OF SAFETY /

Case	Loading Condition	Factor of Safety	Shear // Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with $K_h/K_v = 9$ assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

/ Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

// Other strength assumptions may be used if in common usage in the engineering profession.

* The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. Safety Factors. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \qquad (1)$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. Seepage Failure. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H \gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure, internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluate by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \quad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha(1 + \tan\phi \tan\alpha)} \quad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \quad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} \quad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \quad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \qquad (8)$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \qquad (9)$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.

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